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The Sanitary Engineer Looks Forward

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ANY attempt to present a prospect in the field of sanitation over the next several decades is highly vulnerable, for even under the most favorable circumstances a prophet is literally "without honor in his country." Fully realizing the risks of prophecy, it still seems helpful to let one's imagination range over the field.

Many years ago the late President A. Lawrence Lowell of Harvard summarized the years following wars in these terms (1):

> It is hardly an exaggeration to summarize the history of four hundred years by saying that the leading idea of a conquering nation in relation to the conquered was, in 1600 to change their religion; in 1700 to change their laws; in 1800 to change their trade; and in 1900 to change their drainage. May we not say that on the prow of the conquering ship in these four hundred years first stood the priest, then the lawyer, then the merchant, and finally the physician?

Status of Sanitation

This all too brief summary, with the necessary modifications and expansions appropriate to the complex world in which we now live, is as generally applicable today as it was many decades ago. The damages created by war were paralleled by the realities of the peacetime periods before World War II. A review of the countries of the world prior to 1933 would have disclosed the need for immediate service in the field of sanitation, a need only dramatized by the exigencies of war.

Destruction of water supplies in the Ruhr; the damage to sewage systems in Coventry; the destruction of the water supplies of Manila, either by direct bombing or by deliberate sabotage—re-emphasized the importance of these orthodox necessities of normal living even in the most favored countries. Cholera, typhoid, dysentery, malaria and typhus, only a few of the

scourges of the world, are not only war-borne. They are the normal accompaniments of the peacetime pursuits of man. The insanitary conditions which caused them prevailed before the war and confront us throughout the world after the war. History merely demarks the cyclical rises and falls in their intensities. Wars, depressions and famines awaken the world to this continuance of the struggle of man for protection against his own environment.

It is not an exaggeration to say that Great Britain, France and the United States would all derive advantages from the effective control of yellow fever or malaria. While China or Iran would no doubt benefit from the sanitary engineering experience of the Western world, this does not mean that the Western world itself is living in a Utopia freed of the necessity of controlling the environmental diseases.

This much only can be said, that in those countries blessed with rich resources, with organized public service and with a rapid rate of public works installation, progress in the control of the environmental diseases has been more rapid. How to transmit these blessings to countries or parts of countries historically less favored thus becomes perhaps one of the most important future assignments of the statesmen in sanitation. For India, with its 400,000,000 people; for China, with its almost 500,000,000; for the 250,000,000 Europeans, the 100,000,000 Arabs, the 185,000,000 Russians and the 200,000,000 Latin Americans, the problems of sanitation are essentially the same as those which confronted us in the past and will press us in the future in North America.

The differences in problem throughout the world or within the Western Hemisphere alone are those of timing, of finance, of social organization and public persuasion. Challenges such as these to the sanitary engineer are not limited by political boundaries, by languages, by racial differences or by economic philosophies. They must be met on each ground within the framework of the geographical and political setting. It is still true, as Professor George C. Whipple pointed out many years ago, that "the world is bound in bacterial bonds." To break those bonds requires energy and perspective on a world-wide base and not upon a nationalistic adherence to geographical or political limits of activity.

What Ails the World

To discover future possibilities and necessities for the sanitary engineering function, one need only to turn to a brief review of the diseases that disable and kill in the world. What are they, where do they occur and when may they be conquered with the resources now available? Many years ago the author pointed out that "the ghosts of environmental diseases have been laid in literature but not in fact." Once more war has reminded us of the validity of this observation. The same thought was exemplified by Dr. Afranio Peixoto when he stated in 1941 that "an enfeebled pestilence is not a danger past."

The key to the future activities of the sanitary engineer, therefore, must rest upon the fact, as Col. Dieuaide has so well stated, that "preventive medicine has not yet abolished any disease. The price of health security is eternal vigilance."

The emphasis on these considerations stems from the fact that the superficially minded, the hasty diagnostician and the "nouveau" expert, all

jumping from one new slogan to the next, tend to forget that the victories of the past presuppose continued protections for the future. Control of the environmental disease cannot rest upon such fits and starts of organization or concept.

In a recent discussion of China's health problems, Dr. Szeming Sze points out that, on any one day, 16,000,000 of the 400,000,000 people of China are sick. With a death rate of 25 per 1,000, more than 10,000,000 people die every year. He estimates that, of the 10,000,000 deaths, 4,000,000 are unnecessary, and probably three-quarters of these unnecessary deaths are attributable to the excessive incidence of gastro-intestinal diseases, the infectious cases of infant mortality and pulmonary tuberculosis. No inconsiderable portion of this morbidity and mortality is due to the absence of any of the elementary provisions for the protection of water supply, for the elimination of sanitary wastes or for the control of insect-borne disease.

In India the situation is more than paralleled. The toll taken by the gastro-intestinal diseases, by the diarrheas of children, by the malarias of all ages is staggering in its magnitude and devastating in its restrictions on the opportunity of people to develop their resources to the utmost. So widespread and so dramatic are these tolls in India that, in the 1945 issue of the British *Who's Who,* a distinguished leader in India lists as his recreation: "propaganda for rousing the sanitary conscience of the people in the matters of public health . . . research in intestinal flagellate protozoa and fermentation bacteria." Even allowing for the strong diversions which this intellectual may permit himself, it is a tribute to his recognition of the plight of his people that these efforts in the field of sanitation loom large in competition with the more familiar tennis, walking or swimming.

Conditions, of course, are paralleled in almost every country of the globe where the diseases of filth still reign supreme as the major cause of death and of misery while still alive. Many observers have already pointed out that these problems of environment cannot be disposed of or masked by referring to them as "tropical diseases." They are neither tropical in origin nor restricted by latitude. Historic accident may emphasize their tropical location, but the conditions which engender them cannot be easily or accurately relegated to the tropics.

Colonel Dieuaide properly points out that there are in fact few truly tropical diseases. The diseases so classified are the stock in trade of India and China, for example. Cholera and plague are tropical only because they have been successfully driven into areas not yet capable of wiping them out. By the grace of God, money and sanitary organization, they are rare in Western Europe and the Americas. No one, of course, discounts the relationship of geography to disease, but in the field in which we are today active, the future holds challenges to the sanitary engineer, whose efforts are not limited by conditions in the tropics or the remote areas of the world.

When we compare, for example, the principal causes of death in the United States and in Latin America, we find the major difference is one of time. In 1900, diarrhea-enteritis and typhoid fever were among the 10 leading causes of death in the United States. In 1937, diarrhea-enteritis was still among the 10 leading difficulties. Typhoid fever had been curbed, not abolished,

as will be pointed out later on. In Latin America, however, the six diseases or combinations of diseases which seem to stand out as the principal causes of death are tuberculosis, diarrhea-enteritis, infant mortality, malaria, pneumonia and heart disease.

Perhaps the major health problem of Latin America as a whole is malaria, when its complete incapacitating power is considered. Even though dysentery and the general intestinal parasites do not appear among the six principal causes of death in Latin America, because their killing power is low, they still present one of the major problems in public health because of their profound debilitating effect on individual vitality.

If we review these statistical implications of environmental diseases of the world as a whole, we cannot escape the conviction that the diseases subject to the controls of the engineer will be by far the most important health problem confronting the world for many decades. It is not too much to say that the greatest probable source of accomplishment in the reduction of morbidity and mortality in the world would be through environmental sanitation.

Future Functions of the Engineer

All that has been said above makes it reasonably obvious that the engineer of the future is confronted with both an unlimited task and an engrossing opportunity. The normal functions which he has been called upon to perform in the past are all too familiar. His functions for the future in the fields of water supply, sewage, air control, housing, and insect and rodent control remain essentially the same. Their geographical spread, their rate of application, their expansion in understanding will all need to be speeded up. Perspective will no doubt require an expansion beyond that ever before envisioned. These opportunities in the postwar period are well summarized, even though briefly, in the opening paragraph of the Progress Report of the Committee of the American Society of Civil Engineers on Advancement of Sanitary Engineering, 1946:

> All surveys of the personnel needs of the postwar world indicate an increased demand and expanding field of service for sanitary engineers. In the realm of public health; in the design, construction and supervision of sanitary works; in the armed services; in far-flung relief and rehabilitation activities; in educational work; in many types of industry; in the design, manufacture and sale of sanitary equipment; in the food handling and processing industries; in some fields of municipal government; in the vast and long-neglected realm of housing; and in many other activities—there is increasing need for the services of trained and experienced sanitary engineers.

That there is much to be done everywhere cannot be gainsaid. It is appropriate perhaps to point out that these comments are as pertinent to our activities in North America as they are in Latin America or any other country in the world. It is too early to forget that, in the latter part of 1939 in the Manteno State Hospital of Illinois, 543 cases and 60 deaths occurred in a typhoid fever epidemic. The causes may still appear to be debatable, but strong evidence implicates the water supply of that institution as having been polluted by sewage (2).

During 1943, for example, there were 389 outbreaks of disease in the United States transmitted by food or water. They caused 25,665 cases of reported illness and 56 deaths. The

unreported cases probably number many times those actually listed.

These "ghosts" of the environmental diseases still walk the streets of North America.

The Problem of Money

One of the great difficulties in the control of environmental diseases is that any effective procedure requires money. The pernicious cycle of disease to poverty to disease represents perhaps one of the most important challenges to the sanitary engineer. What are the devices by which environmental structures may be designed, constructed and operated more cheaply than they now are? By what measures is it possible to reduce the unit cost of purifying water, of treating sewage, of controlling insects and rodents?

In some of these fields, the war fortunately has produced possibilities of materials and equipment which are less expensive than those previously available. Some of these have immediate application and use in the control of environmental diseases. The engineer, however, must develop new techniques of financing which would make it possible for governments throughout the world to apply the known solutions to the devastating problems of disease.

Meetings of the sanitary engineers of the Americas will offer an opportunity for much discussion on the problems of financing the installation and the operation of many devices for sanitation. It has long been the author's judgment that too little study has been given to financial procedures. Those matters have been too frequently relegated to administrative officers unfamiliar with their importance or insufficiently ingenious to develop new procedures for paying for sanitary installations. The provision of money is an essential sanitary engineering objective which must be emphasized for speeding up future activity.

What relationship there should be between central and local governments in the provision of money is not only a matter of political philosophy from country to country but also a matter of local availability of funds. In some countries it is no doubt true that most of these activities can be adequately financed through local units of government. In others it is equally obvious that central governments must do much in the next decade to help carry the financial burdens made necessary by sanitation.

The author has no pretension to the ability to point out where these real responsibilities should be lodged. They are mentioned here primarily to focus attention on their key importance to sanitary engineering progress throughout the world.

Administrative Structures

Of almost equal importance in the introduction of sanitary reform in any country is the development of the administrative structures necessary to execute and to operate the installations required. Here, too, political philosophy dominates the scene.

There is perhaps no general law, other than that of expediency, which will determine how much local autonomy and responsibility should prevail in contrast with the responsibilities of the central government. Each country again would probably find the most successful relationships established out of the history of its attributes, its political origins, its racial combinations and its advance along the road to healthy democratic institutions.

In this field, too, the sanitary engineer has a part to play. He must

familiarize himself with the lessons of other countries in administrative management and structure. He cannot, of course, slavishly apply the lessons of other countries to the solution of the problems of his own country. He can, however, suggest and devise administrative structures which would facilitate the purely managerial phases of environmental sanitation. He should be able to share the values to be obtained from central government stimulation with the preservation of local autonomy and responsibility. The countries in Latin America, for example, offer demonstrations over the entire spectrum of central-local government equilibrium. Any permanent group of sanitary engineers should in the course of time introduce objective analyses of those various forms of governmental administrative structures. Out of such analyses should come various examples of how best to establish the governmental machinery for the most rapid and effective introduction of sanitary structures and measures.

Education and Research

To accomplish the objectives all too briefly discussed, the development of sanitary engineering staffs will be required. Such development will place upon the universities of the Americas a task which cannot be lightly treated. Here, too, sanitary engineering groups should in the not too distant future evaluate the requirements, the status and the type of education and training which will be demanded for the most effective practice of sanitary engineering. Suggestions for curricula for accredited schools of sanitary engineering and public health may very properly be one of the early assignments of such groups.

A necessary concomitant of a program of education for professional workers lies in the field of research. Much remains to be discovered in the general field of environmental sanitation. Many investigative tasks are still ahead. Fundamental data on many of our activities are still lacking. The development of essential features of a research program is an additional assignment to serious-minded members of sanitary engineering groups.

Professional Status

All of us are aware that the sanitary engineer not only has a struggle with the physical environment, but has likewise the task of strengthening his capacity and his position in the field of public health activity. This cannot be accomplished by sleight of hand. His status in the future, whether strong or weak, will rest upon the quality of his equipment and of his performance.

One of the best methods of intellectual group-fertilization is in the strengthening of professional groups and organizations. The range of problems which they might with profit discuss and survey covers the status, the professional training, the pay and the public position of the sanitary engineer. All of these will be issues of the future which remain to be argued, measured and promulgated.

Summary

Problems requiring the energies of sanitary engineers are world-wide in character. They are intellectually stimulating and cover a service to the people of the world transcended by no other activity. A necessarily limited review of these problems and of possible solutions has been presented. A scope of activity has been defined for the sanitary engineer of the future,

which can no longer be delimited by the purely technological. Political philosophy, financial program, administrative structure and public education are all essential bases for sanitary engineering action. Technology alone will not bring on the rapid correction of the evils engendered by insanitation.

One cannot escape the essential validity of the argument of one of our contemporary authors (3):

> Let no cultivated reader despise these details (lavatories, sinks, sewers and manholes). There is no truer sign of civilization and culture than good sanitation. It goes with refined senses and orderly habits. A good drain implies as much as a beautiful statue. And let it be remembered that the world did not reach the Minoan standard of cleanliness again until the great sanitary movement of the late nineteenth century.

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Comparison of Formulas for Pipe Flow

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Presented on May 7, 1946, at the Annual Conference, St. Louis, Mo.

The manufacture of large-diameter steel water mains has been notably advanced in the last ten to fifteen years. The riveted pipes of two decades ago have been replaced by welded lines, and the art of lining has been improved. Spun bituminous enamel linings, with "glassy" surfaces, and smooth, spun cement-mortar linings are being widely used. Spun linings are also being used in cast-iron pipe, and surfaces of concrete pipe have been improved by spinning or by the use of new placing methods for poured pipe. Increasing use is being made of cement-asbestos pipe. These innovations are so recent that there is still a scarcity of published flow data for pipes of these types.

In the Fall of 1944 the California Section of the A.W.W.A. appointed a committee on steel pipe coefficients, having as its primary objective the investigation of flow in these relatively smooth pipes. It is contemplated that this will require a considerable program of experimentation.

The committee is not specifically charged with the responsibility of deciding what formula should be used, but rather with gathering data on coefficients. It is, however, considered desirable to look the field over, at least casually, before beginning the investigation; hence the investigation herein described has been conducted. An outline of the research to be undertaken is to be prepared by others.

Summary of Present Situation

The pipe flow formulas available to and being used by water works engineers have been accumulated over a period of some two hundred years. There are many of them, all empirical. They vary in form and yield discordant results. American engineers have gradually narrowed usage to less than a dozen equations, but so far have been unable to agree upon a single "best" formula. There is perhaps no need for such an agreement, but it is well to have some idea of how seriously the various formulas differ.

There is a vague feeling that conventional * pipe flow formulas are "bad." A conception of just how bad they are compared to formulas used for other purposes, why they are bad, and why there is a tendency to mistrust them, may be gained from a very simple comparative example. Suppose it is required to find the area of a steel bar needed to support a specified

* The adjective "conventional" is used to designate formulas and equations in general use by water works engineers prior to, say, 1930.

load in tension. A very simple formula is utilized. Strength is assumed to be proportional to area of cross-section. It should be observed that this is only an assumption, which may or may not be precisely true, but which is accepted without question. The load is simply divided by the breaking strength of the materials of the bar, as found by averaging the results of tests on a number of specimens.

This procedure is simple, straightforward and logical. Everyone understands it, or thinks he does. No one thinks of this procedure as faulty. Yet the area obtained in this manner is multiplied by 3 or 4 or 5, for safety.

The mechanism of resistance to flow of fluids is much more complex. No one understands it. Except for velocities below practical limits, no definite law of friction-velocity-diameter relationship has been evolved comparable in simplicity to the theory of strength-area proportionality for the tension bar. It has not been possible in the past to agree on a formula and confine experimental work to a search for coefficients. Each experimenter has attempted to collect a mass of data, plot it, and fit a curve to the results. Such curves are arbitrary or empirical.

There are many difficulties in the way of constructing a flow formula in this way. The controlling element, theoretically, is surface roughness. As there is no absolute standard of roughness, surfaces are classified on judgment. Such a procedure cannot be exact. Also, there are always valves, manholes, angles in alignment, joint offsets and other irregularities that cause variation in results. And, of course, experimenters make errors and personal equations cannot be eliminated.

For these reasons and others, each investigator is likely to get a new and independent equation. Claims and counterclaims follow, and the impression arises that all of the proposals are poor—as perhaps they are from the point of view of theoretical accuracy. Within limits, however, many of them may be considered adequate for practical purposes until something better is available. At least, it is never necessary to multiply the pipe area computed by these equations by three or four or five, as with the tension bar.

Requirements for Accuracy

Conventional pipe flow formulas are not precise. Neither are other conditions in the problems in which they are used. Although accuracy is a desirable attribute in any mathematical work, much effort can be wasted in carrying computations to a precision not justified by the basic conditions of operation. This is particularly true in pipe design. Water-using communities grow differently or in different places than anticipated, and water sources fall short of or exceed expectations. Rarely can the required flow through a proposed pipeline be foretold within 1 or 2 or even 10 per cent of actuality. Still more rarely can conduit surface conditions be foretold within these limits. The effects of slight bends, irregularities, valves, turnouts and the like introduce further uncertainties, and all pipes are subject to deterioration with age. This deterioration varies with water quality, kind of pipe, type of use and with many other factors. Attempts to foretell time-capacity relationships accurately have not been very effective. Water systems, particularly city grids, grow by "cut and try" and will continue to do so, regardless of the accuracy of capacity computations. Hair-splitting is of no avail. Admis-

sion of these difficulties is not defeatism, but merely a facing of facts, just as is done with the tension bar.

It is of course desirable to use the most accurate computation methods available, and efforts toward the improvement of such methods are worthy and essential. Engineers should keep abreast of the latest developments, and, when new and better methods of computing pipe losses are established, they should be used; but a change to a new proposal should be made with due deliberation.

Recent Experimental Work

During the last two or three decades, a systematic attempt has been made to develop rational flow formulas. This work is still under way in some of the world's best laboratories. Some of the results have been excellent. Leading analysts are devoting much time to the problem. This work is particularly important to mechanical engineers, petroleum engineers and others interested in the transportation of fluids other than water. Conventional formulas apply only to water. Others now being sought are expected to be universal, good for all kinds of pipe and fluids.

The development of some of the new equations will be discussed briefly, their proposed application described, and their results compared with results of conventional methods.

Conventional Equations

No attempt will be made to cover the field of conventional equations completely. Only a few typical formulas will be mentioned; more complete lists and general historical statements can be found elsewhere.

The study will consist primarily of a comparison of formulas by means of diagrams. A few experimental data from published sources will be introduced for illustrative purposes. This report is strictly an appraisal. It contains no new data and no original ideas, but it is hoped that the contemplated investigation will contribute many of both.

Primary Equations

Each of the flow formulas to be considered herein may be reduced to one of the following three primary forms:

Chezy: $$V = C\sqrt{rs} \qquad (1)$$

Darcy: $$H = f\frac{L}{D}\frac{V^2}{2g} \qquad (2)$$

Exponential: $$V = Cr^x s^y \qquad (3)$$

where C is an experimental coefficient, D is pipe diameter, f is an experimental friction factor, g is the acceleration of gravity, H is head loss in a length L, r is the hydraulic radius, s is the slope, V is mean velocity, and x and y are experimentally determined exponents.*

These three primary forms are not independent. Obviously, Eq. 1 is merely a special case of Eq. 3, with x and y each being 0.5. Equation 2 is a rearrangement of Eq. 1, s being replaced by H/L, r by $D/4$, and C by $\sqrt{\frac{8g}{f}}$.

The Chezy formula, Eq. 1, represents the most venerable form, having been developed by Chezy in 1775. Originally, it was probably considered rational, with C constant, or at least constant for a specific surface roughness. Although these assumptions have been found to be untrue, the for-

* All symbols are defined where first used and are listed and defined at the end of the paper.

mula is still in rather wide use, C being treated as a variable and its value computed from auxiliary equations. Recent research indicates that for flows in the higher practical ranges for a fixed value of g, C actually may be constant, not for specific surface roughness alone, but for specific relative roughness, that is, ratio of roughness to diameter.

The Darcy formula, Eq. 2, is also of early origin. It is transformable into Eq. 1, and, if either f or C is known, the other can be found. Of the early forms, this formula is the most frequently used by present-day analysts, largely because f is dimensionless, whereas C in Eq. 1 is the square root of an acceleration. In a fixed gravitational field this is of little practical importance.

The general exponential formula, Eq. 3, is the basis of the Manning, Williams-Hazen and Scobey formulas, all well known and used by American engineers. Values of C and of x and y are determined by experiment. All equations of this type are empirical. Constants are found by plotting experimental data on logarithmic paper and fitting straight lines to them, or by some more elaborate process.

Exponential formulas are generally developed on the theory that x and y should be constant. Each experimenter, however, arrives at different values. It is doubtful if any presently available values are precise or, indeed, if these exponents actually are constant. Allowance for roughness of pipe surface is made in the coefficient C.

Secondary Equations

Many auxiliary equations have been proposed for the determination of the coefficient C in Eq. 1, among which are those of Kutter and Manning:

Kutter:

$$V = \left[\frac{41.6 + \frac{1.811}{n} + \frac{0.00281}{s}}{1 = \left(41.6 + \frac{0.00281}{s}\right)\frac{n}{\sqrt{r}}}\right]\sqrt{rs} \quad (4)$$

Manning:

$$V = \frac{1.486}{n} r^{2/3} s^{1/2} \ldots\ldots\ldots (5)$$

In addition to the notation previously established, n is a factor presumed to depend on surface roughness alone.

In the Kutter formula, Eq. 4, the complicated term within brackets represents C in Eq. 1. This equation is purely empirical. It was created largely, if not exclusively, from data on flow in open channels, but has subsequently been extensively checked against observed flows in pipes. Its value for computing open channel or pipe flow can be determined only by experiment. There is no theoretical basis for its extension beyond the limit of tests.

A disadvantage of the Kutter formula is its complicated form. Practically, this is obviated by the abundance of tables and diagrams available for its solution. A point in its favor to the "practical" mind is that engineers are generally familiar with values of "Kutter's n." Subsequent comparison will show that, within limits, those who prefer this formula may continue its use for water pipes without fear of disaster.

The Manning formula, Eq. 5, may be used as an aid to the solution of the Chezy formula by making use of the fact that:

$$C_m = \frac{1.486}{n} r^{1/6} \ldots\ldots\ldots (6)$$

or it may be solved directly, thus be-

coming a special case of Eq. 3. The statement frequently made that the constant 1.486 is introduced to make Manning's n equal to Kutter's n is misleading. As originally written in metric units, the constant did not appear, the metric form being:

$$nV = r^{2/3}s^{1/2} \qquad (7)$$

The constant 1.486 is merely the cube root of 3.28 (the equivalent in feet of one meter) and is required to make n the same in English as in metric units. Similar adjustments were required in transposing Kutter from metric to English units. The extent to which Manning's n may be considered "the same" as Kutter's n will be revealed in subsequent comparisons.

The Manning equation is readily solved on the ordinary slide rule and hence is more convenient to use than Kutter. Tables and diagrams for Manning are also available but are not essential. Manning's formula, like Kutter's, is purely empirical; its accuracy can be tested only by experiment.

Exponential Formulas

In addition to that of Manning, already discussed, several formulas have been proposed giving specific values to the coefficient and exponents of Eq. 3. The most notable and the most widely used in this country are the Williams-Hazen formula and a group of three developed by Scobey. These equations are used only for water pipe:

Williams-Hazen:

$$V = 1.318C_w r^{0.63} s^{0.54} \qquad (8)$$

Scobey—Wood Stave Pipe:

$$V = 1.62D^{0.65}H_s^{0.555} \qquad (9)$$

Scobey—Concrete Pipe:

$$V = C_s d_i^{0.625} H_s^{0.5} \qquad (10)$$

Scobey—Riveted Steel and Other Pipe:

$$H_s = K_s \frac{V^{1.9}}{D^{1.1}} \qquad (11)$$

or

$$H_s = \nu^{0.1} M_s \frac{V^{1.9}}{D^{1.1}} \qquad (11a)$$

In these equations, C_w, C_s, K_s and M_s are experimental coefficients, H_s is Scobey's head loss per 1,000 ft., d_i is pipe diameter in inches and ν (nu) is kinematic viscosity.

The Williams-Hazen formula, Eq. 8, is perhaps more widely used than any other for the design of municipal water lines in America. It is less popular among irrigation, drainage and hydroelectric engineers or other designers of large water pipes. Although readily solved by logarithms, its routine solution without special aids is tedious. This difficulty is overcome by the use of tables or diagrams. A special slide rule for its solution also is available. This formula has been used successfully for the design of thousands of miles of pipelines, particularly in the 6- to 60-in. size range. It is purely empirical and should not be used outside the range of experimental verification.

Like n in Kutter and Manning, the Williams-Hazen coefficient C_w has a concrete meaning to the practical water man. The constant 1.318 is included to cause C_w to approximate Chezy's C, Eq. 1, on a slope of 0.001. At best, this is an unnecessary complication; worse still, it is usually written as $0.001^{-0.04}$, perhaps for the purpose of revealing its derivation to the elite of the profession who understand that kind of mathematics.

The Scobey formulas, Eqs. 9 (1), 10 (2) and 11 (3), are widely used in irrigation work. They were developed by F. C. Scobey of the U. S. Dept. of

Agriculture between 1910 and 1930 from a large number of field tests. Many of these tests were made by Scobey himself; the remainder were gathered from other available sources.

These three equations were derived as separate problems at distinctly different times, in the order listed, which accounts, at least in part, for their divergent appearance. Each of them represents a particular solution of Eq. 3, with the constants in each determined for the class of pipe to which the formula is supposed to apply. Equation 11 can be converted to the form of Eq. 3 by solving for V.

Williams and Hazen, in Eq. 8, made the exponents constant for all kinds of pipe, all variation for surface differences being taken care of in C_w. Scobey has changed x and y from class to class of pipe. The wood-stave pipe formula, Eq. 9, is "fixed," there being no variable coefficient.

Equation 11, the last evolved, covers a wide variety of surface conditions from heavily riveted pipes to smooth interior metal pipes of various kinds. It was developed with considerable care and has a touch of "modernity" in that the sum of the indices, $x + y$, Eq. 11, is 3.0. This was supposed by Scobey to cause M_s, in the alternative Eq. 11a, to be dimensionless. This theory has subsequently been questioned (4). Equation 11a also introduces an allowance for temperature change through the factor $\nu^{0.1}$. Equation 11 is presumed to apply to a standard temperature of 15°C. (59°F.).

Dimensional homogeneity is not of great importance in an empirical equation of limited range, and allowance for temperature change is not important in ordinary water pipe work. In his steel pipe bulletin (3, p. 81), Scobey gives a table of velocity corrections for temperature variations from the standard value of 15°C. For a range of 11°C. (approximately 20°F.) above and below this standard, the table shows a variation in V of plus and minus 1.5 per cent. The corresponding variations in s or f, with V constant, are minus and plus 3 per cent. Whether these corrections should be applied in a given case depends on circumstances. Usually they are overshadowed by other uncertainties.

Further consideration is given to temperature effects in the discussion of more recent formulas.

Each of Scobey's equations represents with reasonable accuracy the data from which it was derived. Each in itself is easily diagrammed.

Comparison of Conventional Formulas

Standard of Comparison

Before proceeding to more modern developments, the equations thus far presented will be compared with each other. To accomplish this without unnecessary duplication, it is desirable to choose a standard formula with which the others may be compared. The Williams-Hazen is the most widely used of the pipe formulas and is perhaps as reliable as any. Scobey's steel pipe formula, Eq. 11, however, is the latest to be evolved, was carefully worked out and has more specifically prescribed values for its coefficients. Consequently, Eq. 11 is taken as a standard of comparison and each of the others is compared with it. Finally, Kutter and Manning are compared directly with each other.

As will subsequently appear, formulas equivalent for one range of conditions are not necessarily equivalent

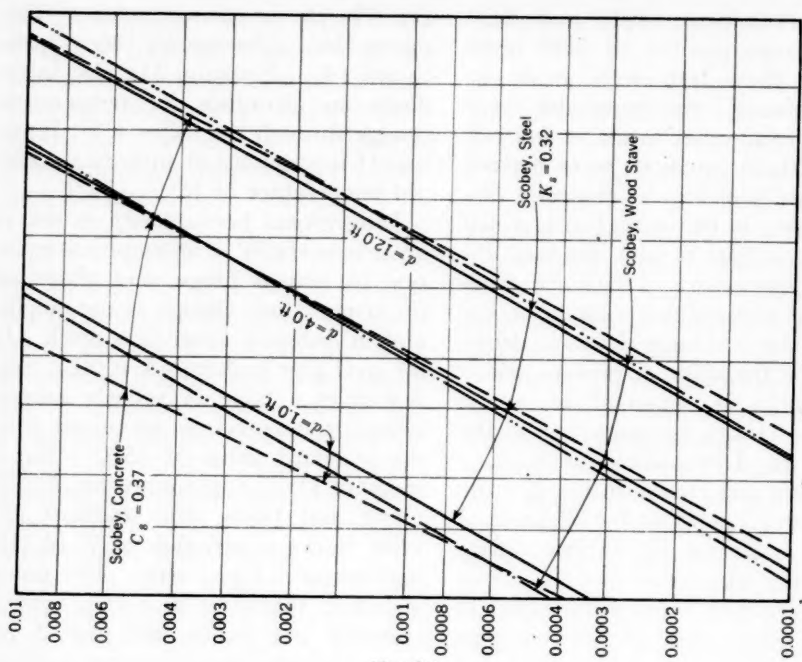

Fig. 2. Comparison of Formulas: Scobey to Scobey

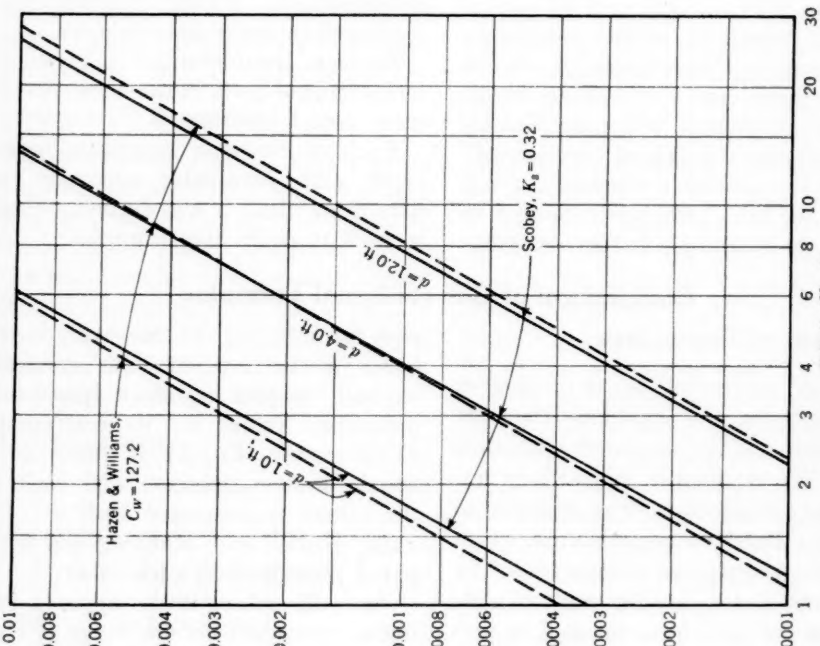

Fig. 1. Comparison of Formulas: Scobey to Williams-Hazen

for others. Any attempt at universal comparison runs into complicated diagrams. Consequently, except for Kutter vs. Manning, comparison will be made for three sizes of pipe: 1 ft., 4 ft. and 12 ft. in diameter. A value of $K_s = 0.32$, Scobey's recommendation for "continuous interior" pipe, will be used in Eq. 11, and values of the coefficients n and C in other equations will be chosen to give equivalent results for $D = 4.0$ ft., $V = 6.0$ fps., this being assumed to represent average conditions. Additional diagrams should be made by anyone interested in a wider range of application.

Scobey vs. Williams-Hazen

The relation of Scobey's steel pipe formula, Eq. 11, with $K_s = 0.32$ to the Williams-Hazen formula, Eq. 8, is illustrated in Fig. 1. To secure coincidence with $D = 4.0$ ft. and $V = 6.0$ fps., C_w must be given a value of 127.2. This is a higher value than many designers use for "smooth interior" pipe of the class covered by Scobey (3), but is well below the optimum that might be expected from a new, perfectly installed, commercially smooth pipe. In other words, it is moderately conservative. Interest here lies only in its comparison with $K_s = 0.32$.

An examination of the two curves for $D = 1.0$ ft. shows a variation, especially for low velocities, well outside of the 1 or 2 per cent usually considered mathematically desirable. Anyone familiar with the plotting of "field data" derived from numerous sources, however, would be surprised if the plotted points fell exclusively on either of these curves or even between the two of them. Agreement for the 12.0-ft. pipe is better.

The only means for determining which of the two equations is better is a detailed comparison with experimental data. To date, such comparisons have been inconclusive, due, no doubt, to the uncertainties inherent in the problem. In view of these uncertainties and the fact that C_w and K_s are never known within the precision indicated, it may be concluded that there is at present no practical basis for a choice between Eq. 8 and Eq. 11.

Scobey vs. Scobey

In Fig. 2, Scobey's steel pipe formula, Eq. 11, is compared with his concrete pipe formula, Eq. 10, the coefficient of the latter being adjusted to equal the former at $D = 4.0$ ft. and $V = 6.0$ fps. Reasonable conformity is shown in the 4.0-ft. to 12.0-ft. diameter range, but there is a measurable discrepancy for the 1.0-ft. pipe. The 1.0-ft. pipe curves could be brought into approximate coincidence by changing coefficients, but this would throw the others off. There is no obvious reason why surfaces which give equivalent results at one diameter should not do the same at all diameters even though one be steel and the other concrete.

Equation 9, for wood-stave pipe, being (presumably) for a single class of surface, has a fixed coefficient which cannot be logically adjusted to a standard. As written, however, it coincides almost exactly with Eq. 11 for $K_s = 0.32$, at $D = 4.0$ ft. and $V = 6.0$ fps. Consequently, the curves for wood-stave pipe are added to Fig. 2, and show better agreement with the standard than do those for concrete. They do not generally approach the 1 per cent or less agreement which the theorists quite properly seek, but they are probably within the "scatter" of observations on actual field installations of water pipes.

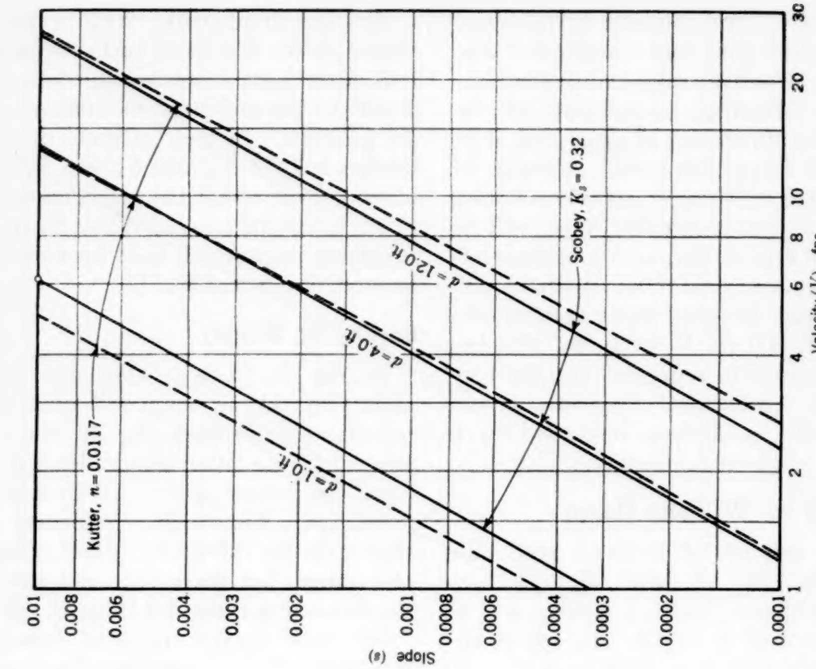

FIG. 4. Comparison of Formulas: Scobey to Kutter

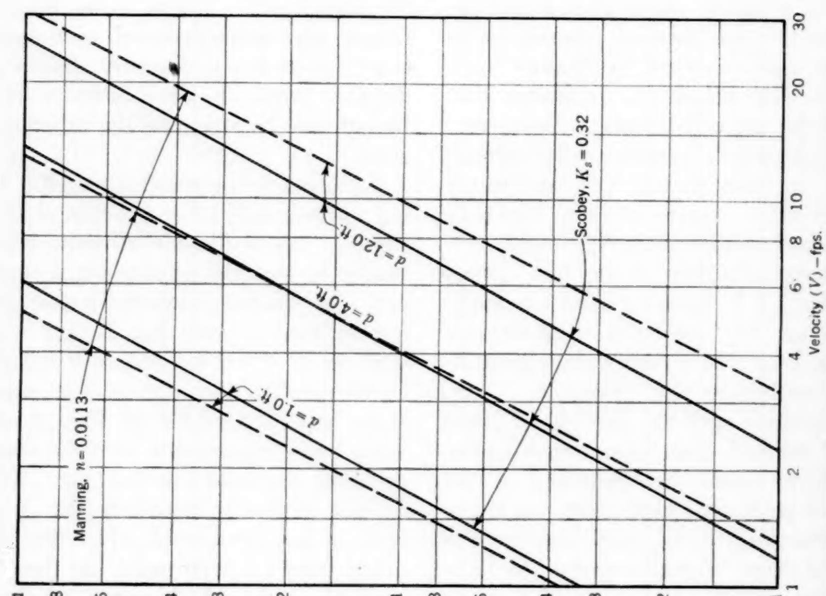

FIG. 3. Comparison of Formulas: Scobey to Manning

Scobey vs. Manning

The relation of Eq. 11 ($K_s = 0.32$) to Eq. 5 ($n = 0.0113$) for 1.0-ft., 4.0-ft. and 12.0-ft diameters, is shown in Fig. 3. The slope of the curves being fairly similar, agreement is fair throughout for the 4.0-ft. diameter. Agreement is not good for the 1.0-ft. pipe and is bad for the 12.0-ft. diameter.

Scobey vs. Kutter

Kutter, Eq. 4, is equivalent to Eq. 11 with $K_s = 0.32$, at $D = 4.0$ ft. and $V = 6.0$ fps., when n is 0.0117. Curves for 1.0-ft., 4.0-ft., and 12.0-ft. pipes based on this value of n are shown in Fig. 4, together with corresponding curves for Eq. 11.

Contrary to popular opinion, the agreement is somewhat better than for Manning. It should be observed that values of n are different on Figs. 3 and 4, which accounts for the difference in position of the Kutter and Manning lines for $D = 12.0$ ft.

Bearing in mind all of the uncertainties involved, there is no overwhelming choice between these formulas within the range of Fig. 4. Scobey is no doubt to be preferred. Neither should be extrapolated beyond the range of experimental confirmation.

Values of n used in Figs. 3 and 4 are higher than would be expected from experimental tests on new, smooth interior, straight pipe ideally installed. Values as low as 0.010 or 0.0105 would not be surprising, and a value of 0.011 might be used for design under favorable conditions although 0.012 is frequently used. This simply indicates that, compared to usual procedure with Manning's and Kutter's formulas, a value of K equal to 0.32 in Eq. 11 is reasonably conservative for continuous interior pipe.

Kutter vs. Manning

Because of the frequent discussions of the relative merits of Kutter and Manning, a direct comparison of Eqs. 4 and 5 is made in Fig. 5. To cover more completely the wide range of conditions to which these formulas are applied in open as well as closed channels, Fig. 5 differs in form from Figs. 1–4. It shows the relationship of C and r (Eq. 1) for both Kutter and Manning for three values of n. A very smooth surface ($n = 0.010$), an intermediate one ($n = 0.015$), and a relatively rough one ($n = 0.030$), are chosen for the dual purpose of covering a wide range and avoiding the confusion of overlapping lines in the diagram. The co-ordinates of the diagram are logarithmic; hence Manning, for each value of n, plots as a single straight line. Kutter, not being exponential, is represented by curved lines. Also, because Kutter's C involves both s and r, a family of curves is required for each value of n. Curves for $s = 0.001$ and 0.0001 are shown. The limits of the figure are purposely exaggerated to show trends.

It may be observed from Fig. 5 that:

1. The curves (for a given n) all cross at $r = 3.28$ ft. (1.00 m.).
2. Kutter and Manning are reasonably concordant for small smooth conduits and for large rough ones.
3. They are discordant for large smooth conduits, small rough ones, and both small and large medium rough ones.

This diagram (among many others) was originally prepared by Harold P. Vail of the Los Angeles office of The Metropolitan Water Dist. of Southern California as a guide for the selection of a formula for the design of lined

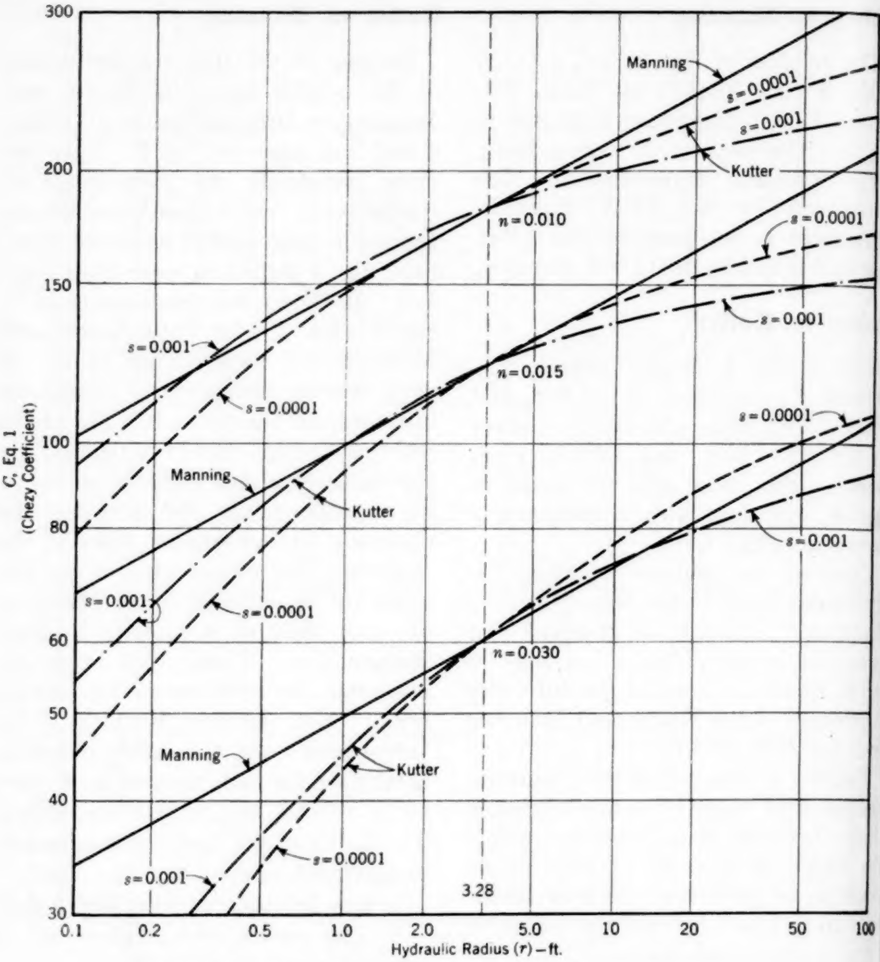

FIG. 5. Comparison of Formulas: Manning to Kutter

waterways on the main Colorado River aqueduct for diameters ranging from 9.0 ft. to 16.0 ft. Because of the proximity of r to one meter (3.28 ft.), there was little choice between Manning and Kutter so far as results were concerned. Manning was chosen on the basis of convenience in use.

For values of r running down to 0.25-ft. (for a 12-in. pipe), the comparison is not favorable unless the surface condition is very good. It is not evident from this comparison, however, which formula is best.

A few well-placed experiments on *identical surfaces* in the range $r = 0.1$ ft. to $r = 3.28$ ft., with a few more for large conduits, would go far toward settling the relative merits of these two equations. No opportunity to secure such data should be overlooked, as these equations are still in wide use for open channels. In making such tests, it is essential that surface condi-

tions remain identical. In existing "field data" * this certainty is lacking.

Obviously, Kutter and Manning are not universally equivalent. Either can be used with confidence only within the range of experimental verification.

Most modern textbooks condemn, without reservation, the use of Kutter for the design of pipes. King (5) devotes several pages to the discussion of the relative merits of Kutter and Manning (for open channels) and decides in favor of Manning. Numerous other writers find reasons for questioning the validity of Kutter both for open channels and pipes. It is particularly criticized as a pipe-flow formula because it was confected from open channel data. This particular argument is not too logical from an empirical point of view. The author, however, has no desire to make a case for Kutter.

As previously stated, the only way to judge the relative merits of empirical flow formulas is by comparison with the result of experiment. This has been done almost *ad infinitum* with Kutter and Manning, but without definitive results, largely due to lack of range and definiteness in data.

Recent Trends

The Need

Conventional equations are lacking in rational foundation and apply only to the fluid, or fluids, for which tested, usually water. Except for Scobey's Eq. 11a, no allowance is made for viscosity or fluidity, which changes with temperature. For water works design, the effect of temperature change on flow resistance is rarely if ever important. It is of paramount importance, however, with heavy oils, other viscous liquids and gases. Consequently, those interested in the handling of such materials are seeking a better basis of design. Recently, much effort has been directed toward the development of a set of dependable, accurate, universally applicable flow formulas. Progress has been made and formulas are now being offered for general use. Consistent data on coefficients applicable to practical water works conditions are still scarce. Whether the time has arrived for abandoning the conventional formulas in favor of these new ones must be decided by each individual for himself. It is hoped that this article and the discussions that will follow its publication may help in reaching such a decision.

The New Equations

The most recent ideas are embodied in a set of equations designed to give values of f in Eq. 2. These formulas, being universal, must provide for viscosity. To maintain dimensionality, viscosity must be introduced through a dimensionless parameter. Such a parameter, well known and widely used for other purposes, is the Reynolds (6) number, expressed algebraically thus:

$$\mathbf{R} = \frac{VD}{\nu}$$

where ν (nu) is the kinematic viscosity of the flowing fluid. Adequate descriptions of the Reynolds number and viscosity are available in current texts on hydraulics. The set of formulas follows:

Poiseuille—Streamlined Flow:

$$f = \frac{64}{\mathbf{R}} \quad \text{..........(12)}$$

* The term "field data" is used to designate the results of observations on service installations as opposed to laboratory installations.

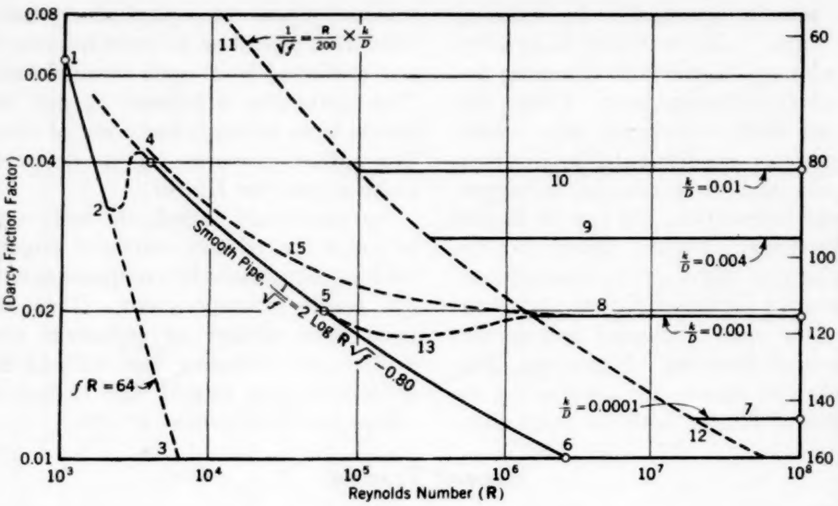

FIG. 6. Elements of f–**R** Equations

von Karman, Nikuradse—Smooth Pipe, Turbulent Flow:

$$\frac{1}{\sqrt{f}} = 2.0 \log \frac{\mathbf{R}\sqrt{f}}{2.51}$$
$$= 2.0 \log \mathbf{R}\sqrt{f} - 0.8 \ldots (13)$$

Nikuradse, Prandtl, von Karman—Rough Pipes, Complete Turbulence:

$$\frac{1}{\sqrt{f}} = 1.14 - 2 \log \frac{k}{D} = 2 \log \frac{3.7}{\frac{k}{D}} \quad (14)$$

Colebrook—Rough Pipes:

$$\frac{1}{\sqrt{f}} = -2 \log \left(\frac{\frac{k}{D}}{3.7} + \frac{2.51}{\mathbf{R}\sqrt{f}} \right) \ldots (15)$$

In these equations, k is the height of the roughness projections or some linear dimension representing the height, form and distribution of such projections. All logarithms are to base 10, constants having been adjusted to permit this change from natural logarithms.

The relationships of these equations to each other and to the flow problem in general are illustrated graphically in Fig. 6, the logarithmic co-ordinates of which are f and **R**. The problem is divisible into three distinct stages separated by two transition zones.

Laminar Flow

The first stage covers streamlined or laminar flow in smooth or rough pipes. For this stage, f is computed from Eq. 12 which plots as the single straight line, 1–2–3. Equation 12 is rational and universal. It applies to all kinds of pipes and to all fluids. A full discussion of streamline flow can be found in any recent text on hydraulics or fluid mechanics. It is of no importance in this paper. It is not controversial and the very low velocities involved never need to be considered in the design of a water system.

Turbulent Flow, Smooth Pipe

At about point 2, Fig. 6, the streamline formula breaks down and the flow becomes turbulent. The second stage

turbulent flow in smooth pipe, is represented by Eq. 13 which plots as a single curve, 4–5–6. The ending of curve 1–2–3 and the beginning of curve 4–5–6 depend on a number of variables and may be considered uncertain. The transition between these curves is represented by a curve such as 2–4, uncertain as to position and form. Because of the extremely low velocities involved, flow in the region 2–4 is of no importance to the water works engineer.

Rough Pipe

Although Eq. 13, curve 4–5–6, applies primarily to smooth pipe, it also appears to apply to rough pipes through a limited velocity range (7, 8). With increasing **R**, a break occurs into a transition zone, eventually passing to some horizontal line such as 7, 8, 9 or 10 in Fig. 6, depending on the relative roughness of the pipe, that is, the ratio of the height (or some function of height, spacing and arrangement) of the roughness projections to the pipe diameter.

The curves 7, 8, 9 and 10 represent plots of Eq. 14, each for a particular value of $\frac{k}{D}$. They are independent of **R**. The nature of the transition between the smooth pipe curve and the rough pipe curves 7, 8, 9 and so on, is uncertain. Nikuradse (8) has shown rather definitely that for smooth pipes roughened with sand grains of uniform size, the break-over for a particular relative roughness follows some such curve as 13, Fig. 6. There are, of course, similar connecting curves for other horizontals.

The position of the curve 11–12, which marks the limit of the zone of uncertainty and the beginning of the horizontal lines, is not exactly known. In fact, it probably is not a definite line. Its approximate location, however, is given by Moody's equation (9):

$$\frac{1}{\sqrt{f}} = \frac{\mathbf{R}}{200} \times \frac{k}{D} \qquad (16)$$

Other equations for its approximate location are available (4).

General Formula, Turbulent Flow

The equation for curve 13 is unknown. If it were established that the break is always of this form, it would be on the side of safety to ignore it in practice. The horizontal lines could be extended to intersect the smooth pipeline, the smooth pipe curve being used to the left of the intersection.

"Natural roughness," however, appears to differ from Nikuradse's artificial roughness, and experiments indicate that the change-over in commercial pipe from Eq. 13 to Eq. 14 follows somewhat the form of curve 15, Fig. 6. The equation of curve 15 is not known. Colebrook (4), however, has proposed a formula, Eq. 15, to bridge this gap. This formula covers, in fact, the whole range of turbulent flow. It is asymptotic to curve 4–5–6 on the left and to the various horizontal lines (such as 8) on the right. To the right of 11–12, the term containing **R** may be ignored.

Eq. 15 can hardly be classed as entirely empirical; neither can it be classed as rigidly theoretical. Its practical accuracy in the transition zone is well supported by tests. It is reasonable in appearance and has been received with much hope. Additional data on natural as well as artificial roughnesses are needed.

Discussion of the *f*–R Formulas

It is not practicable in this paper to show the derivation of Eqs. 12–15. A

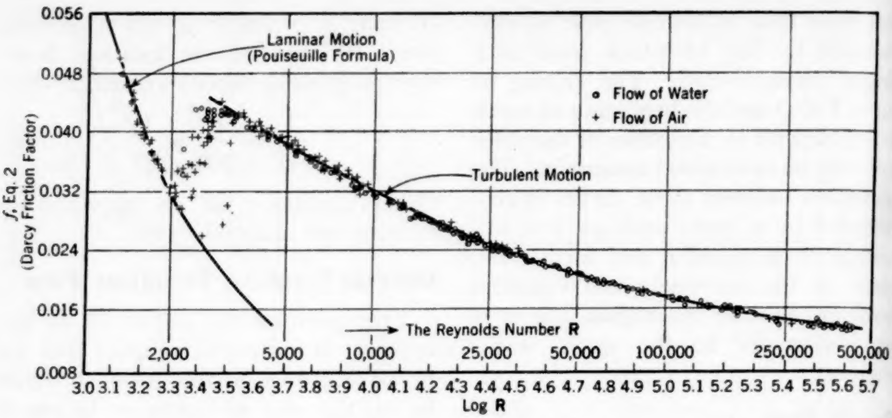

FIG. 7. Basis for Eq. 13

brief outline of their origins, however, can be attempted. More complete information will be found in the sources listed in the bibliography.

Equation 12, originated by Poiseuille (10) in 1842, was confirmed by the experiments of Reynolds (6) in 1883 and has later been shown to be fully rational. It may be considered exact but unimportant to the water works engineer.

Equation 13 was evolved in basic form by von Karman (11) in 1930. It was later slightly adjusted and rearranged by Prandtl and others (12, 13) to put it into closer agreement with Nikuradse's (14) experimental data.

This equation has a mathematical basis and is frequently referred to as "exact" or "theoretically correct." It is derived from von Karman's theoretical equation for velocity distribution in pipes (7, 11). That it is fully rational may be subject to some question, but its substantial accuracy for a wide range of conditions is established. Its mathematical derivation was inspired by a remarkable experimental diagram first published by Stanton and Pannell (15) in 1914. This diagram has been copied many times in various forms. It was based on laboratory tests with both air and water in small drawn pipes of lead and brass. An arrangement of the data by Bakhmeteff (7), with the notation adjusted to this paper, is reproduced as Fig. 7.

A similar diagram reaching further into the practical field, published by Colebrook (4) in 1939, is presented in Fig. 8. Most of the "scatter" of plotted points are for centrifugally spun cement-lined cast-iron pipes in large sizes, for which the consistency of small drawn brass and lead pipes cannot be expected. Figures 7 and 8, however, offer staunch support for Eq. 13.

Because f occurs in both sides of Eq. 13, it is tedious to solve algebraically. This has led many writers to propose simplified approximations, some of which agree with the original within 1.0 per cent or 0.5 per cent to very high values of **R**. Most of these simplifications are just as good as the original for all practical purposes. As routine solutions are made by curves or tables, however, the need for these simplifications is negligible. For this reason, none of them has been quoted here.

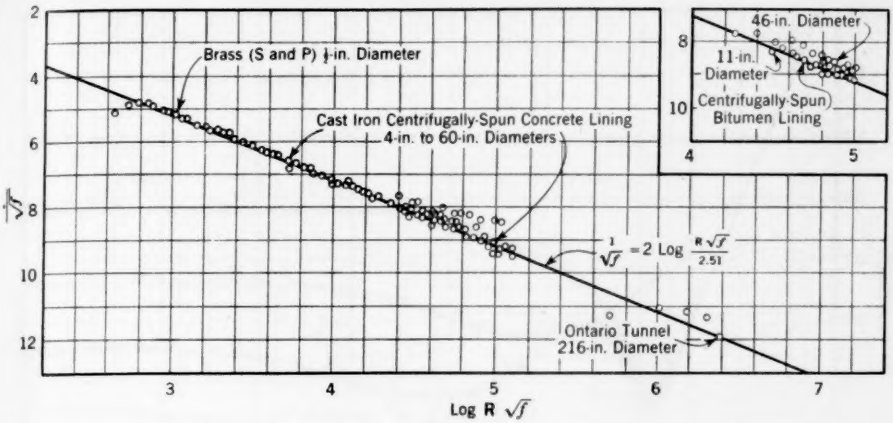

FIG. 8. Verification of Eq. 13 on Smooth Commercial Pipe

Equation 14 was devised to conform to tests by Nikuradse (8) on smooth pipes roughened by sand grains of uniform size glued to the interior surfaces. The remarkable consistency of these data is shown in Fig. 9, which is taken with altered notation from Fig. 24 on page 35 of Bakhmeteff (*7*). This diagram furnishes the basis for curves 7, 8, 9 and 10, and corresponding transition curves such as curve 13 of Fig. 6. Equation 14 applies only to the region to the right of curve 11–12 of Fig. 6. The substantial accuracy of this equation for the types of roughness covered by Nikuradse's experiments seems established. There are also reasons for believing that other rough pipes behave similarly to the right of curve 11–12, Fig. 6.

There is so far no means of specifically determining k for "natural" roughness. Such roughness is completely miscellaneous and perhaps forever will defy evaluation by micrometer measurements. Reliance must be placed on hydraulic tests by which a particular surface may be assigned a value of k, just as Kutter's n or the Williams and Hazen C_w is determined. The influence of joints, bends and other special items will require special consideration just as they do for conventional formulas. Additional and more careful observations on full-sized pipes from which such values may be computed are needed.

Equation 15 was devised by Colebrook (4) to cover the transition from the smooth to rough pipe laws for pipes of natural roughness. The performance of such pipes *in the transition region* appears to differ from that for Nikuradse's (8) pipe. Plotted for a specific relative roughness, it takes the form of curve 15, Fig. 6. Actually it replaces both Eqs. 13 and 14, approaching these two equations asymptotically to the left and right, respectively. Although based upon much rationalization, this equation can hardly be said to be classified as theoretically pure. Colebrook finds it in reasonable conformity with tests on pipes of natural roughness. An example is shown in Fig. 10, copied from Fig. 8, page 151 of Colebrook's paper on turbulent flow in pipes (4). The plotted points for wrought-iron pipes are as uniform as might be expected with pipes of this kind. Colebrook presents similar data for other classes of pipe.

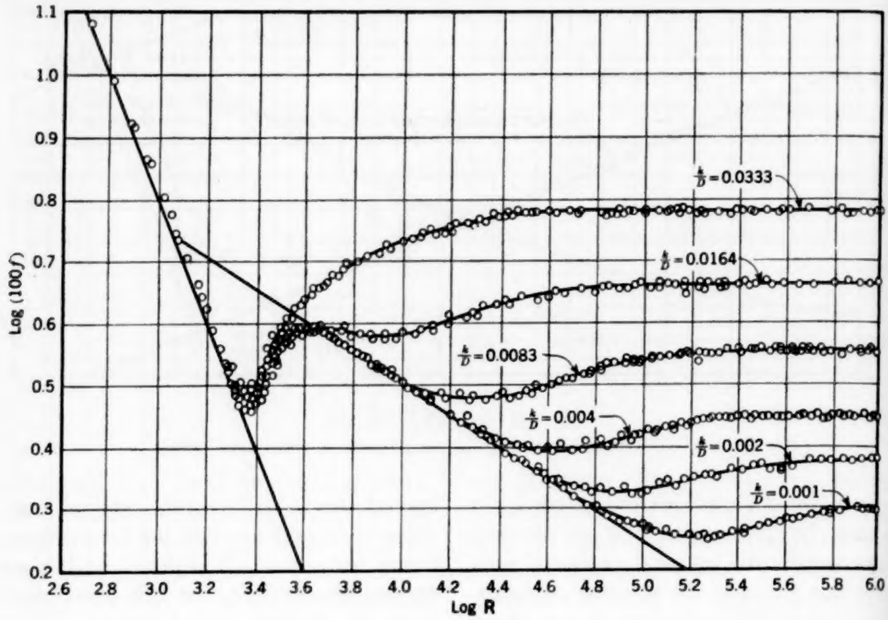

FIG. 9. Nikuradse's Sand Grain Roughness

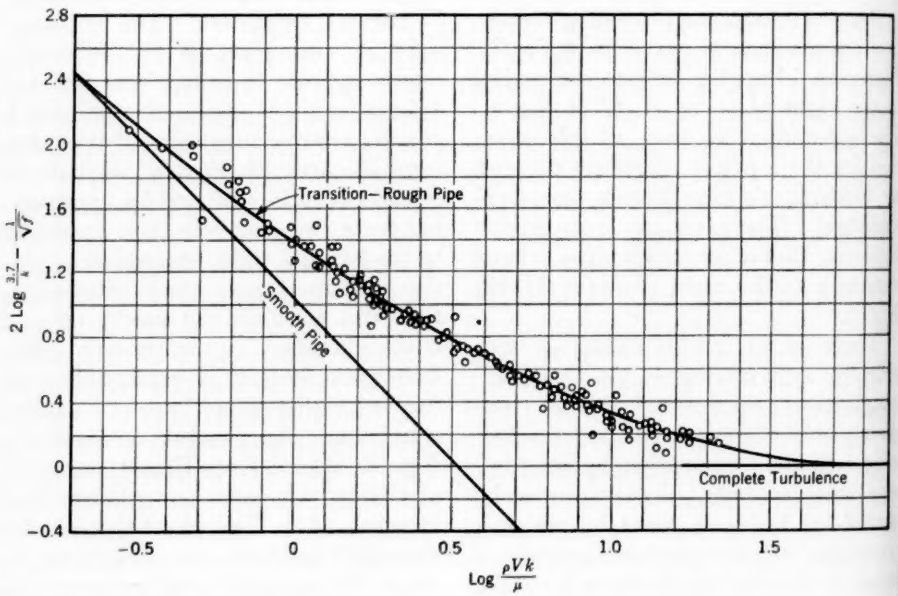

FIG. 10. Deviations From Colebrook Formula for Rough Pipe

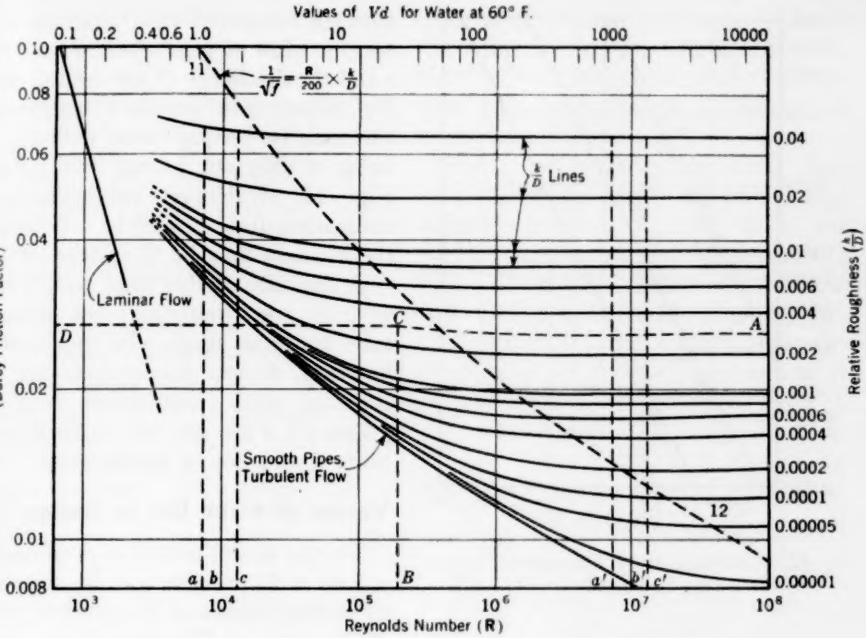

Fig. 11. Solution of Colebrook Formula for Rough Pipe

There are other equations, old and new, for the determination of f, but those enumerated seem to express the present trend of thought.

Utilization of the f-R Formulas

If and when Eq. 15 is fully established as applicable to commercial pipes and values of k are determined for various types of surfaces, this equation should replace all others. It will not be universal and all-embracing until the discrepancy between Nikuradse's and Colebrook's roughnesses is correlated, but the presumption is that it may ultimately be shown applicable to all cases of natural roughness.

Its logarithmic form may appear forbidding. It is tedious to use in routine computation, but no more so than Kutter. It is readily solved by diagrams, as is clearly shown by Moody (9). In Fig. 1 of his paper, Moody presents a working scale diagram of the general form of Fig. 11, which operates very simply. (Fig. 11 is not drawn to a working scale.) A known value of k is divided by a trial (or known) value of D to get a value of $\frac{k}{D}$, say as at A. Knowing (or assuming for trial) values of V and ν, a value of **R** is computed as at B. Then by passing along the $\frac{k}{D}$ line to C, vertically above B, the value of f at D may be read. If preferred, an auxiliary scale may be supplied from which the Chezy experimental coefficient C, instead of f, may be read. Such a scale is shown at the right of Fig. 6. Also, for a mean temperature and a specified fluid, an auxiliary scale for VD may be used in place of **R**, as shown at the top of Fig. 11. The diagram can, of course, be drawn to co-ordinates of C

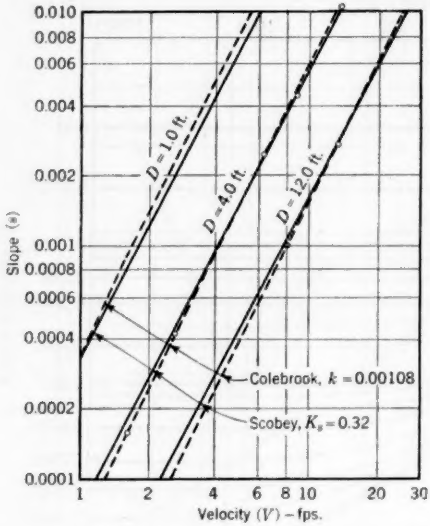

FIG. 12. Comparison of Formulas: Scobey to Colebrook

and **R** if desired. The VD scale on Fig. 11 is drawn for d in inches.

Most of the pipe design work in any particular office centers around a few standard types and it is customary for the designer to have available tables or diagrams for use with each specific type. Thus, diagrams may be kept handy for Scobey's wood-stave, concrete, welded steel, riveted steel (of specific roughness) or similar diagrams for other formulas. Equation 15 is readily adaptable to such a procedure. For a specified value of k, the $\frac{k}{D}$ lines of Fig. 11 become D lines.

One of the practical disadvantages of Eq. 15 for routine design of water pipes, as compared to formulas for Chezy's C and the exponential formulas, is that it requires the computation of two auxiliary quantities, the Reynolds number and the velocity head. These quantities are readily determined but in oft-repeated routine, a diagram like Fig. 11 is at a disadvantage compared to a diagram for a specific class of pipe where s and V or s and the discharge Q are used directly. By assuming a standard temperature this may be accomplished with Eq. 15, using a diagram having the form of Fig. 12, developed subsequently in comparing Eqs. 11 and 15. If desired, the abscissa may be Q instead of V.

A diagram of this kind would apply to only one roughness, one temperature and one fluid. In most offices, however, design assumptions are held constant, and constructing such diagrams for a few sets of standard conditions would not be burdensome.

Values of *k* for Use in Design

Moody makes tentative proposals for values of k, presumed to be equivalent to certain values of Kutter's n, giving P. Panagos of Princeton Univ. as authority (9); these values are given in Table 1. Such comparisons are not altogether logical, as k and n are not subject to being universally equated.

Values of k for specified classes of pipe surfaces are given in Table 2, which is based on Moody's Fig. 2 (not reproduced here). Commenting on the values for concrete, Moody states that they:

. . . may be somewhat more definitely described as follows:

0.001 0.003	0.0115 0.0125	Highest practical grade of concrete. Surface and joints smooth
0.01	0.014	Concrete surface with slight form marks, fairly smooth joints or roughly troweled
0.03	0.016	Prominent form marks or deposits of stones on bottom.

These values are presumed to have been derived largely from data listed in the various bulletins by Scobey (1, 2,

TABLE 1

Suggested Equivalent Values for k *and* n

Kutter Coefficient of Roughness (*n*)	Absolute Roughness (*k*)
0.0105	0.00015
0.011	0.0005
0.012	0.002
0.013	0.005
0.014	0.011
0.015	0.02
0.016	0.03

TABLE 2

Values of k *Given by Moody*

Class of Pipe Surface	*k*
Drawn Tubing	0.000,005
Commercial Steel or Wrought Iron	0.00015
Asphalted Cast Iron	0.0004
Galvanized Iron	0.0005
Wood Stave	0.0006 to 0.003
Cast Iron	0.00085
Concrete	0.001 to 0.01
Riveted Steel	0.003 to 0.03

3). The method of derivation is not revealed. As will be shown subsequently, Scobey's formula is not directly comparable with Colebrook's. These proposed values need further checking.

Temperature Effect, *f*–R Equations

Temperature effect enters the *f*–**R** equation through the influence of viscosity on **R**. As Eq. 14 is independent of **R**, flow for the condition of complete turbulence is not directly influenced by temperature. Indirectly, it determines the value of VD, above which complete turbulence exists.

The influence of temperature on smooth pipe flow can be illustrated by referring to Fig. 11, assuming that it is desired to investigate the effect of a ± 20°F. change on flow water through a pipe designed for a temperature of 60°F. For a specified value of VD, **R** will vary with $\frac{1}{\nu}$. For the temperatures assumed, the variation will be in the proportions: 60,000 : 82,000 : 108,000.* These ranges are very considerable. The variation in *f*, however, is much less.

For a low velocity, or small pipe, such that **R** is 10,000 for 60°F., corresponding values of **R** for 40°F. and 80°F. are, respectively, 7,300 and 13,200. These three values are noted at *a*, *b*, and *c*, on Fig. 11. Corresponding values of *f* for smooth pipe (by computation or from a larger and more exact diagram) are given in Table 3.

TABLE 3

Effect of Temperature Change Upon Friction Factor (f) *of Smooth Pipe*

Temp. °F.	Reynolds Number (**R**)	Friction Factor (*f*)	Percentage Change
40°	7,300	0.0336	109
60°	10,000	0.0309	100
80°	13,200	0.0287	93

It is obvious from Fig. 11 that these differences in *f* decrease with increasing relative roughness of pipe, becoming negligible at the top of the diagram.

Differences in *f* for the same difference in temperature decrease with increasing **R**, as is obvious from Fig. 11. In the vicinity of **R** = 10,000,000 (see *a*′, *b*′ and *c*′) the range for smooth pipe, points *a*′, *b*′, and *c*′, is given in Table 4. For full turbulence with rough pipe

TABLE 4

Effect of Temperature Change Upon Friction Factor (f) *of Rough Pipe*

Temp. °F.	Reynolds Number (**R**)	Friction Factor (*f*)	Percentage Change
40°	7,300,000	0.00849	105
60°	10,000,000	0.00810	100
80°	13,200,000	0.00779	96

* Values taken from a viscosity table.

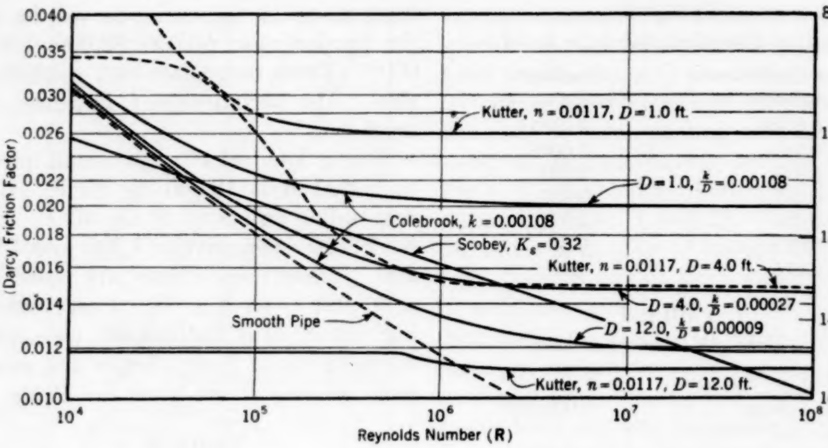

FIG. 13. Comparison of Formulas: Scobey to Kutter to Colebrook

—that is, above the line 11–12, Fig. 11—the effect of temperature is zero. For small values of **R**, the temperature effect in Eq. 15 is greater than that previously noted for Scobey. For large values of **R**, it is smaller.

Whether temperature should be considered for incomplete turbulence in rough pipes depends on the precision sought. For most practical water works problems, temperature effect is overshadowed by more important variables and may be ignored. The average temperature variation from a standard of 60°F. is not likely to equal the 20°F. used in the foregoing computations. A careful record of temperature conditions, however, should never be omitted from experimental observations.

Comparison of f–R and Conventional Formulas

Colebrook vs. Scobey

Comparison of f–**R** equations for finding f for use in Eq. 2, with more conventional procedures, may be accomplished by returning to Scobey standards for 1.0-ft., 4.0-ft. and 12.0-ft. pipe diameters, with constants equated at $D = 4.0$ ft. and $V = 6.0$ fps., at a temperature of 60°F. The comparison is first made in an f–**R** type of diagram, Fig. 12. At constant temperature, Eq. 11 (Scobey) plots as a single straight line on this diagram *for all sizes* of pipe. At $D = 4.0$ ft. and $V = 6.0$ fps., **R** is approximately 2,000,000 and at this value of **R** the Scobey curve passes approximately through $f = 0.015$; and from Eq. 15 $\frac{k}{D} = 0.00027$. For a 4.0-ft. pipe this corresponds to a value of 0.00108 for k.

This is well removed from Moody's value of $k = 0.00015$ for commercial wrought-iron or steel pipe. As noted in comparing Scobey with Williams-Hazen and Kutter, however, $k_s = 0.32$ is conservative for smooth new welded steel pipe. As Fig. 13 is for mechanical comparison only, it makes no difference at what level the comparison is made, but it does not follow that the proper evaluation of k can be neglected.

The impression might be gained from Fig. 12 that Eq. 11 (Scobey) and Eq

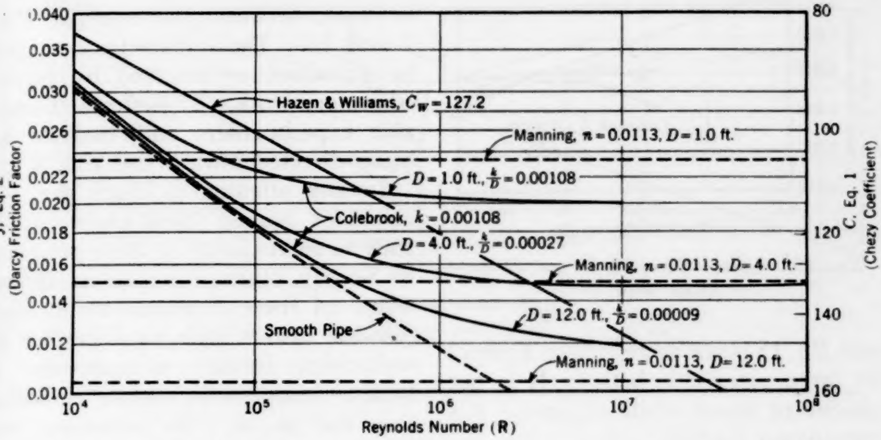

FIG. 14. Comparison of Formulas: Colebrook to Williams-Hazen to Manning

15 (Colebrook) are hopelessly divergent. Scientifically, this is true. Within moderate limits, however, final results are no more discordant than between some of the conventional formulas previously compared. This is more apparent in Fig. 12 which corresponds in form to Figs. 1 to 4, and is of less sensitive scale than Fig. 13. Within the range of Fig. 12, Scobey and Colebrook agree better than Scobey and Manning, Fig. 3. Nevertheless, variations in s for fixed velocity are as much as 20 per cent. The velocity variations for fixed gradients are smaller.

Figures 12 and 13 in themselves offer no basis for choosing between Eqs. 11 and 15. Equation 15 is without doubt nearer to the true form. Equation 11 was deduced arbitrarily from observed data with no pretense to rationality. Equation 15 has much reasoning as well as testing back of it.

Colebrook vs. Kutter

As a matter of interest, the Kutter lines from Fig. 4 are also shown in Fig. 13. Although Kutter plots very erratically on the f–**R** diagram, it is notable that for small pipes ($r < 3.28$ ft.) it agrees with Eq. 15 in form better than does the Scobey formula.

Colebrook vs. Williams-Hazen

If the exponent of s, Eq. 8, is slightly changed from 0.54 to 0.543, this equation, if transposed to the form of Eq. 11, satisfies the dimensional requirement that $x + y = 3.0$ and hence, for a fixed temperature, plots as a single straight line on the logarithmic f–**R** diagram as shown on Fig. 14. This line is plotted for $C_w = 127.2$, the value used in Fig. 1. The discrepancy with Eq. 15 is about the same as for Scobey, Fig. 12, but, as the curve is steeper, any conformity comes with slightly smaller values of **R**.

Colebrook vs. Manning

Figure 14 is also utilized for comparing Eq. 5 ($n = 0.0113$) and Eq. 15 ($k = 0.00108$). As Manning's C varies only with the hydraulic radius, both C and f are constant for a given pipe size and a specified value of n. Therefore, the 1.0-ft., 4.0-ft. and 12.0-ft. pipe curves on Fig. 14 are simple horizontal lines. For small pipes, its conformity

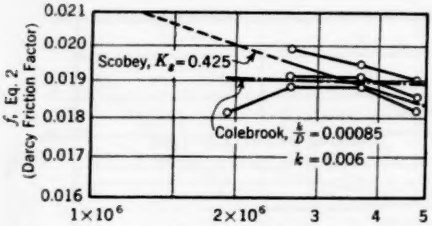

FIG. 15. Scobey Tests 72, 74 and 75 on 84-in. Pipe

with Eq. 15 is not as good as is Kutter; for large pipes it is better. It is not considered worth while to pursue this comparison further.

Examples of Experimental Data

Data shown on Figs. 7, 8, 9 and 10, and on many similar diagrams from sources listed in the bibliography, definitely tend to substantiate the *f*–**R** formulas. These data appear to be well authenticated, and the processes of analysis applied to them are appealing. There is still some lack of harmony, however, between, for example, Figs. 9 and 10. These discrepancies must be explained or resolved by further laboratory work. The efforts of many able experimenters and analysts are being directed toward this end.

In any attempt to check the *f*–**R** equations against "field experiments," such as the majority of those presented by Scobey, other discrepancies appear. Many of these doubtless result from physical causes such as imperfect installations, inexact measurement of losses, variations in surface classification and so on. An occasional test with a reasonable claim to authenticity, however, cuts across the *f*–**R** lines with a definiteness that at least requires explanation. A few such cases will be cited merely to demonstrate that the problem is not yet fully solved.

In addition to the disabilities already mentioned, field measurements on an individual conduit usually cover a very limited range of velocities. Such a case is illustrated in Fig. 15, which

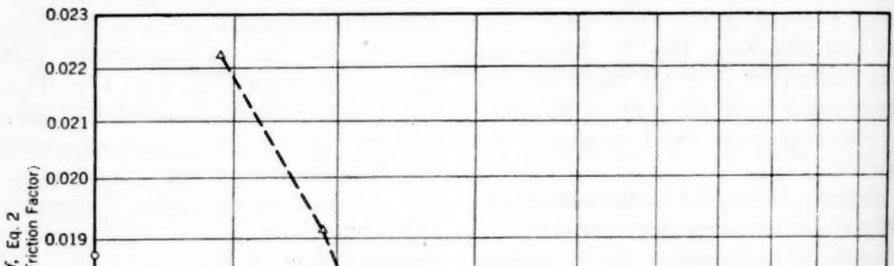

FIG. 16. Scobey Test 78 on 108-in. Pipe

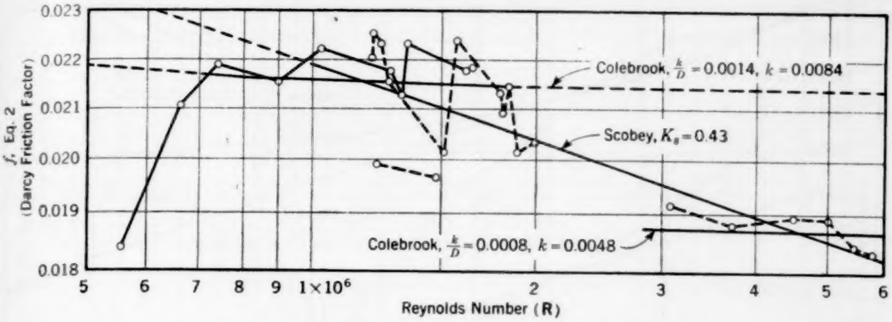

Fig. 17. Scobey Tests 64, 65, 67 and 69 on 72-in. Pipe

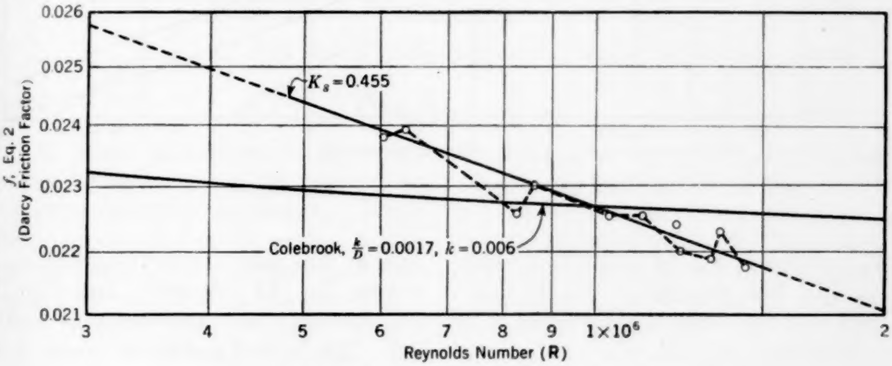

Fig. 18. Scobey Test 46 on 42-in. Pipe

represents three sets of tests by Scobey (3) on 84-in. riveted steel pipe. The results are inadequate for determining whether Eq. 11 or Eq. 15 has the better form. They can be used, however, for determining spot values of K_s or k. There are many such sets of tests which have been analyzed for values of K_s. By plotting as in Fig. 15 or otherwise reducing, they can be used for the determination of additional values of k. A finding that values of k for a specific class of surface is more constant than K_s (or than C_w or n) would indicate superiority of Eq. 15, and of course conversely. A great difficulty is the uncertainty about the equality of surface conditions in different pipes of presumably similar materials. This difficulty occurs even in the same pipe for tests at different times. Variations in K_s or k due to formula deficiency may be overshadowed by variations due to undetermined physical differences. Until means for overcoming such difficulties are found, it is important that any new experiments intended to be used for checking form of equation should cover the widest possible range on each individual reach tested, and should be made in the shortest practicable time.

A slightly greater range of **R** is covered by the data of Fig. 16, also from Scobey (3), for a 108-in. riveted steel pipe. Excluding the two left-hand points which Scobey graded as of "B" quality, these data indicate values of

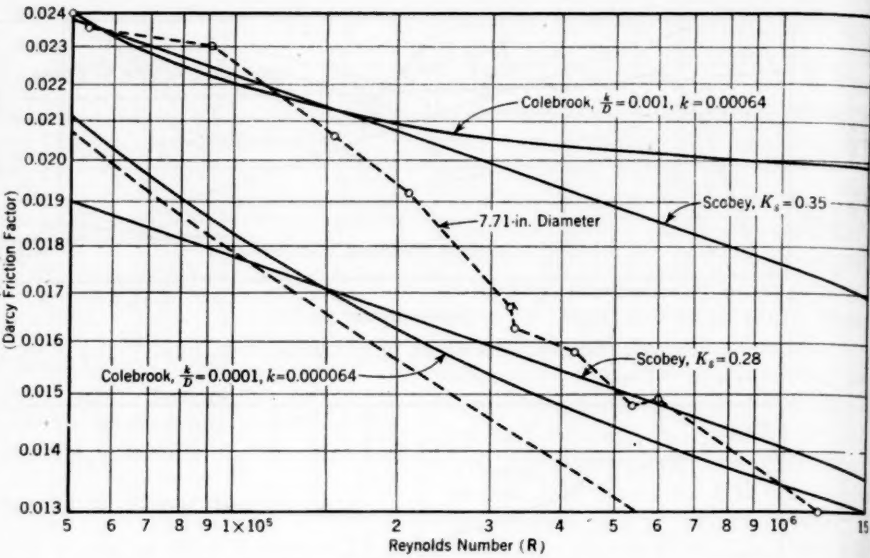

Fig. 19. Scobey Test 2 on 7.71-in. Pipe

$K_s = 0.368$ in Eq. 11 and $k = 0.00405$, Eq. 15. The slope favors Eq. 11 (Scobey), but the range is too narrow to be conclusive.

A range in **R** approximating 1 : 10 for 72-in. diameter riveted steel pipe is shown in Fig. 17. These points were plotted from Scobey's bulletin on riveted pipes. Ignoring the two left-hand points which were for fairly low velocities, they appear to support the slope of Eq. 11 (Scobey). The points are for several groups of pipes in New Jersey, Utah and California, however, and involve many incidental losses in addition to pure pipe friction. They of course show nothing conclusively.

A fairly consistent set of points for a 42-in. riveted steel pipe, again from Scobey, is shown in Fig. 18. These points favor Scobey in their slope but again are of too limited range to be conclusive.

An interesting example for a small riveted pipe, Scobey's test No. 2, is shown in Fig. 19. These data fall within that part of the transition zone where Eq. 11 (Scobey) and Eq. 15 (Colebrook) are approximately parallel. The plotted points cut across both of them. The tests were made by Darcy in 1851. This pipe is within the size range used in the tests upon which the f–**R** formulas are based. It is below the size of average interest to water works engineers.

A set of points for an 8-in. wrought-iron pipe, dipped in asphalt, is shown in Fig. 20. These data were taken by Freeman (16) in 1892 under laboratory conditions. They fall within a range where Scobey and Colebrook are roughly parallel and fit either of them reasonably well, favoring Colebrook slightly.

These examples could be continued at great length. Most of them are deficient in some respects. The few presented show that the problem is not yet solved and illustrate the need for range, consistency, and accuracy in future tests.

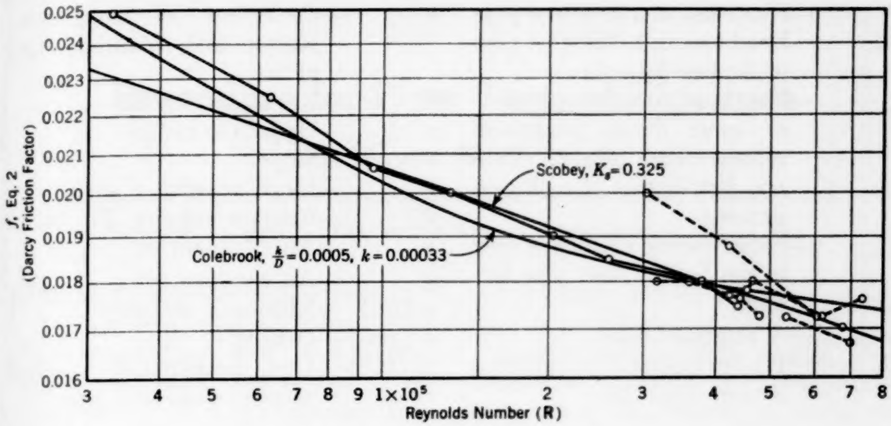

Fig. 20. Freeman Test on 8-in. Pipe

Conclusions

The foregoing discussion leads to certain thoughts which can perhaps be classed as tentative conclusions. From the point of view of the water works engineer and for the purposes of this study, they may be listed as follows:

1. The older pipe flow formulas are imperfect and are limited in scope, but uncertainties involved in their use are not markedly greater than other uncertainties inherent in the design of water supply systems.

2. Of the newer formulas, that for laminar flow may be accepted as theoretically exact and that for smooth pipe as accurate beyond practical needs. These formulas are of little use in the design of water systems.

3. The Colebrook formula for turbulent flow in rough pipes is ideal in its conception, but it requires further substantiation, particularly in the transition zone, and the development of a definite scale of values for its roughness factor, k. At present, practical uncertainties involved in its use are probably greater than for the older formulas. This conclusion applies only to water systems, not to pipes carrying gas or oil and other fluids, for which the old formulas are useless.

4. The forthcoming research program should be directed first toward the collection of friction data on the new types of pipe, suitable for use in either old or new formulas; and second, toward the development of information for substantiating or improving the new universal equations, or for developing better ones.

List of Symbols

C —Experimental coefficient in Chezy formula

C_m —C as computed for Manning formula

C_s —Experimental coefficient in Scobey concrete pipe formula

C_w —Experimental coefficient in Williams-Hazen formula

D —Diameter of pipe

d or d_i —Diameter of pipe in inches

f —Friction factor, Darcy formula

g —Acceleration of gravity

H —Head loss in a length of pipe

H_s —Head loss in 1,000 ft. of pipe in Scobey formulas

k —Height of surface roughness or some linear dimension representing height, form and distribution of such projections

K_s —Experimental coefficient used in the Scobey steel pipe formula

L —A length of pipe

M_s —Experimental coefficient, Scobey Eq. 11a

n —Coefficient of roughness Kutter and Manning formulas

R —Reynolds Number

r —Hydraulic radius

s —Slope

V —Mean velocity

V_f —Friction velocity, Eq. 16

x —An exponent of r

y —An exponent of s

ν (nu) —Kinematic viscosity

ρ (rho) —Density

τ_0 (tau) —Unit longitudinal reaction between fluid and pipe

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Procedure for Flow Tests of Pipelines

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FROM the discussion by Hinds (1), it is apparent that an extension of present pipe flow data is desirable. Such an extension will of course require a definite testing program. It is to the writer and his associates in the Los Angeles Dept. of Water and Power that this problem of planning the necessary tests has been assigned.

One of the first tasks involved in the problem will be the selection of a series of installations suitable for testing. Some preliminary thought has been given to this question, but to date there are no definite results to report. This is a matter that will require the cooperation of the entire water works profession. Selections must be made on the basis of suitability of pipe type, length of reach available for testing, suitability of operating conditions, equipment and personnel available and other pertinent factors.

Water works men who have knowledge of installations suitable for testing by their own personnel or others are urgently requested to report them to the California Section committee on steel pipe coefficients. To assist in deciding what installations are suitable, it is proposed to prepare a manual, or body of rules, setting up requirements and procedures. This discussion is intended as a tentative summary of such a manual, submitted for consideration and discussion. When completed and approved, such a manual will of course be used as a guide, rather than as a rigid control.

In the average water system, ideal conditions for making flow tests seldom exist. For this reason, it seems desirable to develop a manual of procedure so that all data procured will be of value when correlated with tests made in other water systems.

The preparation for a properly conducted test will usually require much more time than the observations themselves. Arrangements must be made in advance for controlled flow conditions. The personnel conducting the work must be thoroughly instructed and all details of the work assigned to definite individuals. A misunderstanding on the part of one individual may easily nullify the value of the entire test.

Selection of the Test Reach

The test reach should be as long as possible, and the type of pipe included must be uniform throughout its entire length with no change in diameter or appreciable difference in the condition of the pipe from one end to the other. There should be the fewest possible specials, no non-closable laterals, no sediment in dips and no negative pressures. The installation records and the pipe itself should be checked thoroughly for the location of fittings such as gates, regulators, reducers, laterals, bends or any other factors which might

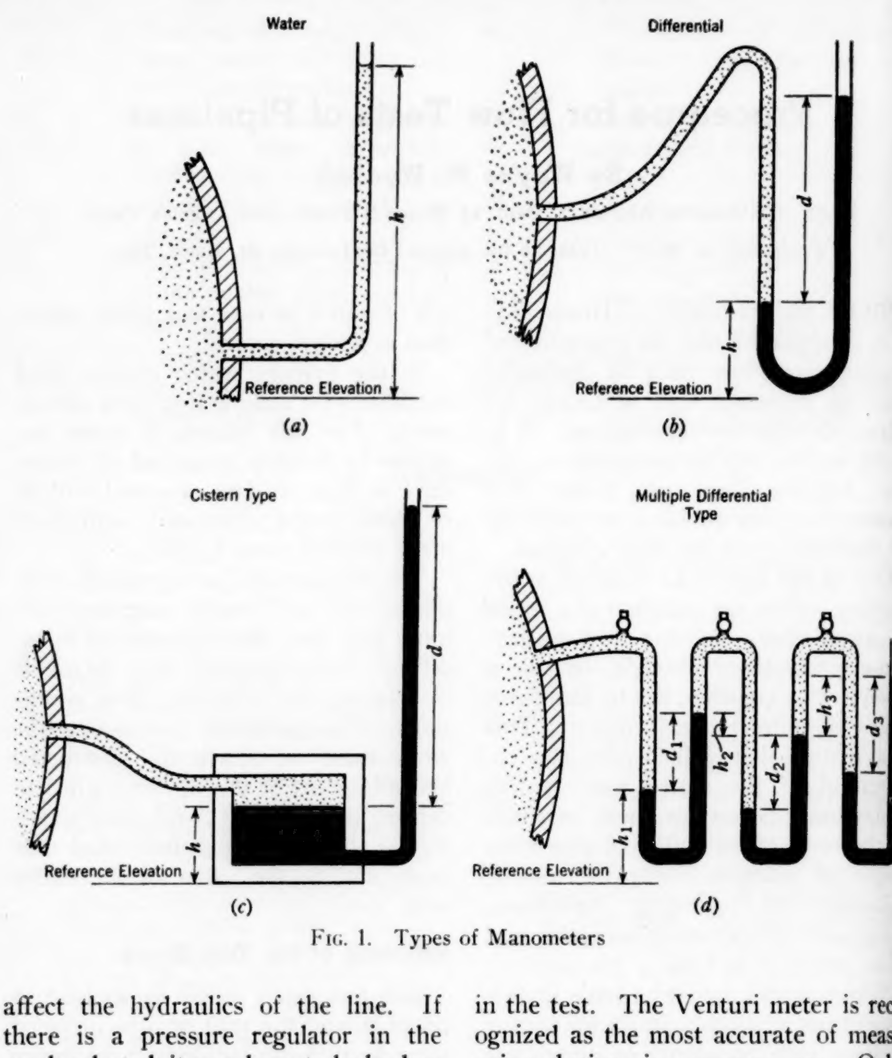

FIG. 1. Types of Manometers

affect the hydraulics of the line. If there is a pressure regulator in the reach selected, it may have to be broken into two sections. The reach should be considered acceptable, however, if the regulator is blocked open and a differential manometer is connected upstream and downstream to determine the head loss through it.

Discharge and Velocity Measurements

The accurate determination of the rate of flow is a very important factor in the test. The Venturi meter is recognized as the most accurate of measuring devices in common use. Only rarely, however, will a Venturi meter be so fortunately located that it will be available for flow determinations in the particular reach of pipe to be tested. If the pipe discharges directly into a tank or reservoir, a volumetric determination can be made by measuring the time required to discharge a measurable volume into the reservoir.

The velocity type meters, while not considered as accurate as the Venturi,

are acceptable if in good adjustment. Where no meter is installed in such location that the discharge in the line can be determined, it may be necessary to use pitot tubes for direct velocity determination. Since pitot tubes are inserted through corporation cocks which can normally be installed without much difficulty, the pitot tube should be a fairly convenient method. Other possible methods such as orifices and weirs are possible only when the line discharges freely into some reservoir or open canal. The use of the salt velocity method, while considered quite accurate, can seldom be used in water works lines since the injection of salt or other chemicals into a line serving water for domestic consumption is not usually desirable. Any measuring device should be carefully checked before the test is made.

Head Measurement

Where the pressure head is not too high above the surface of the ground, the simplest and probably most accurate type of water level indicator is the plain water manometer (Fig. 1a). This type of manometer does not require any elaborate installation. Transparent plastic tubing is ideally suited to this purpose since its flexibility permits easier handling without the hazard of breaking encountered in the use of glass tubing. The gage board may be an ordinary surveyor's level rod set to such an elevation that the piezometric level will fall within range of the graduations.

If the pressure head is so high above the ground as to make the use of the water manometer impracticable, it will be necessary to use mercury or some fluid of higher specific gravity than water. Carbon tetrachloride and bromoform are two commonly used manometer fluids which have specific gravities between those of water and mercury. Mixtures of carbon tetrachloride and pure naphtha can be used to obtain various specific gravities between 1.00 and 1.60. Some coloring matter, such as an oil dye, added in very small amounts to these fluids will not change their specific gravity a significant amount and will facilitate the observations.

The mercury manometers may be either the ordinary U-tube (Fig. 1b) or the cistern type (Fig. 1c). The cistern type manometer is essentially the same as the ordinary U-tube but its construction is such that only one long tube is necessary. The mercury well or cistern has a cross-sectional area many times greater than that of the manometer tube, so that the drop in fluid elevation in the cistern is very small compared to the rise in the tube. If the relative cross-sections of the cistern and manometer tubes are carefully determined, it will be necessary to read only the manometer tube in making observations. The drop in elevation of the mercury in the cistern can be calculated from the rise in the small tube. When properly designed and calibrated, this type of manometer is more convenient than the U-tube.

Where the piezometer level is so high above the ground as to make it impracticable to use a single mercury manometer, several differential manometers can be placed in series (Fig. 1d), and the total head will be that indicated by the sum of the separate readings. The number of manometers so used should be as small as possible since the error increases with the number of separate readings.

Mechanical gages are not desirable as their accuracy is always questionable, even immediately after testing.

The elimination of pulsations in pressure is difficult in many cases. These oscillations in surface elevation may be reduced considerably by enlarging the tube in the vicinity of the surface elevation by some method such as attaching an inverted bottle, with the bottom removed, at the top of the tube. Arrangements should be made for raising and lowering this enlarged portion of the tube to keep the water surface within it. For mercury manometers, or others where the enlarged tube is not practicable, a stop cock may be installed just outside the connection to the pipe, for throttling the pulsations, if required.

Piezometer openings of less than $\frac{1}{16}$ in. in diameter are not recommended, since they are subject to clogging. An opening smaller than $\frac{1}{16}$ in. also results in excessive throttling, and the observer may be led to believe that the fluid level in the manometer has reached equilibrium when it really has not. The openings should have parallel sides, that is, they should be of uniform diameter for a distance of at least twice the diameter of the opening and should be flush with the inner wall of the pipe. The burr must be carefully removed from the inner edge.

The piezometer should not be installed where there is any chance of eddies in the pipe, or immediately downstream from an obstruction such as a gate, bend, pipe joint, or even a rivet head. A distance of approximately 20 diameters downstream from the nearest fittings such as a gate or bend is recommended. A paper by Allen and Hooper (2) on the results of experiments on piezometer openings of different sizes and shapes reports satisfactory results on openings of diameters between $\frac{1}{16}$ in. and $\frac{27}{32}$ in. for a pipe diameter of 12 in. and velocities

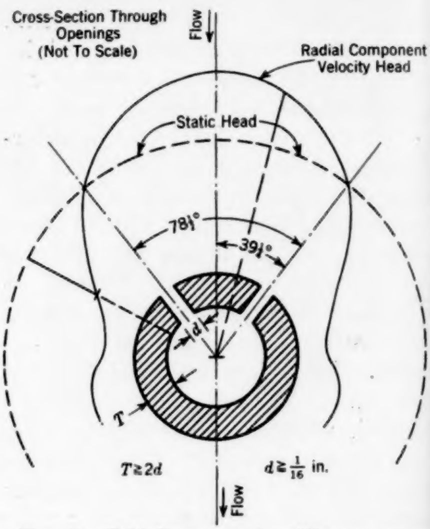

FIG. 2. Self-Compensating Piezometer

up to 7.3 fps. A diameter of $\frac{1}{16}$ in. is recommended unless it is found to result in excessive throttling.

A very satisfactory type of piezometer may be prepared from a tube of circular cross-section by drilling two holes which are centered 78½ deg. apart on the same circumference of the tube. A tube so perforated and normally penetrating a moving stream will register the true static head when rotated to a position in which the 78½-deg. angle included between the centers of the perforations faces and is bisected by the direction of flow. If the tube is rotated to any other position, the head registered will be less than the true head, so that to determine the true amount it is only necessary to get the maximum reading on the tube (Fig. 2). These tubes can be inserted into the pipe through a standard corporation cock by use of a suitable stuffing box, in a manner similar to that used with the pitot tube.

One very distinct advantage of this type of piezometer over the type with

the opening flush with the pipe surface is that it can be inserted to any position in the pipe cross-section and not be affected by velocity head. If inserted so that the openings are near the center of the pipe, the readings are not affected by turbulence caused by small irregularities in the pipe surface. For the lack of any recognized name, this type of piezometer is referred to as the "self-compensating type," since it eliminates the effect of velocity head.*

Fire hydrants are convenient fittings to which manometers can be connected and, under certain conditions, are acceptable. If the connection to the main is made by tees which are identical at both ends of the test reach, any error in one due to turbulence in the vicinity of the tee should be of nearly the same magnitude as in the other and will cancel out in computation of the head loss.

A less desirable connection than the fire hydrant is a customer service, but it must be certain that the corporation cock does not project beyond the interior surface of the pipe. If a customer service is used, it is essential, of course, to check it for leaks, as a relatively small leak would produce a head loss between the point of observation and the street main.

Obviously, the longer the test reach, the less important is the accuracy of the piezometer, since the percentage error in the final results, due to an error in pressure head determinations, varies inversely as the length of line tested.

Whatever type of piezometer is used, it is essential that air bubbles be eliminated from the tubing between the openings and the fluid surface in the manometer. The error caused by air bubbles can be quite large.

The temperature of the water in the manometer tubing may become considerably different from that in the pipe. If the range of elevation between the piezometer opening and the fluid surface in the manometer is large, the corrections for difference of specific gravity of the water in the pipe and in the tubing will be appreciable and should be determined. The temperature in the tubing will be approximately the temperature of the atmsophere, but, where the range of elevation is large, the temperature should be determined on completion of the observations by bleeding the water from the tubing into a vessel and measuring its temperature. Frequent wetting of the manometer and tubing during observation period with water drawn from the pipe will keep the temperature difference small.

Preparation for the Test

If the pipe is large enough and a shutoff is possible it should be inspected on the inside. Photographs should be taken of the interior of the pipe and the inside diameter measured carefully. These measurements should be taken 90 deg. apart and should be of sufficient number along the pipe to assure the observer that the mean diameter has been determined with sufficient accuracy. Silt deposits, blisters, rough spots and the general condition of the interior should be noted. When pipes are too small to permit a detailed

* The self-compensating type of piezometer has been developed as a modification of the "direction-finding pitot tube" (BINDER, R. C. & KNAPP, R. T. Experimental Determinations of the Flow Characteristics in the Volutes of Centrifugal Pumps. Trans. A.S.M.E. **58**: 650 (1936); also, DAUGHERTY, R. L. *Hydraulics.* McGraw-Hill Book Co., New York (4th ed., 1937), p. 170). The direction-finding pitot tube has a partition between the two piezometer openings so that the differential between them can be determined. For use as a self-compensating piezometer, this partition is omitted.

inspection of the interior, the outside circumference should be measured carefully, and the thickness determined by drilling through the pipe in several places. It is highly desirable to cut a coupon from the pipe, so that it can be inspected in detail for the size of projections with reference to the diameter of the pipe. Photographs of the interior side of the coupon should be submitted with the test data.

It is necessary to arrange for shutoff of all laterals, and experienced gatemen should be used to reduce the possibility that some gates may not be completely closed, thus nullifying the value of the test.

The bench marks or reference points for piezometer installations can be established by surveyor's levels, or, if a shutoff of the line is feasible, the bench marks at the piezometer points can be established from the static level of the water while not flowing, as shown by the manometer.

All gates in the line must be completely open. In the case of partial obstructions which cannot be eliminated, but which will produce considerable head loss, the head loss should be determined by differential or U-tube manometers connected to piezometer openings on each side of the obstruction, the downstream one being at least 20 diameters from the obstruction.

It is important that all such data as the type and spacing of joints, description of pipe, location and type of fittings and also the quality of water normally carried in the line be reported. All this type of information is usually available in the records so that its inclusion on the data sheet will involve little additional investigating. A plan and profile of the line showing location of fittings will be helpful in analyzing the results of the tests.

Observations

In conducting observations, the first problem is the establishment of steady flow. It will usually be found that pressure surges persist for considerable lengths of time after steady flow has apparently been attained. This is a matter which must be carefully watched. It is sometimes impossible to eliminate these surges completely. Unless properly handled, pressure surges may seriously affect results, as they do not occur simultaneously at upstream and downstream stations. All observers should have their watches synchronized so that simultaneous observations can be taken at previously arranged intervals of time. Where surges persist, the maximum and minimum elevations should be recorded, together with the exact time. The average elevation of maximum and minimum may be accepted as the actual piezometric level.

Where readings are taken at a considerable distance above the ground, the physical comfort of the observer is not an unimportant detail. Readings are easily missed because of fatigue of the observer.

Velocity traverses of the cross-section of the pipe as recommended by Hinds (1) should be obtained where feasible. These traverses will require pitot tube installations (3, 4).

Tests of each pipe reach should cover as large a range of velocities as possible, with a desirable minimum of three. Where only three velocities are tested, magnitudes in the vicinity of 2, 5 and 8 fps. give a satisfactory range, but it is important to test velocities as high as operating conditions will permit.

Temperatures of the air, the water in the pipe, the water in the manometer tubing and the fluid in the manometer should be taken at the time of the test.

It is suggested that a test data sheet listing the categories of information which will be useful in proper analysis and interpretation of the test data be used. This sheet requires information on:

1. By whom the data are submitted (institution, address and name and title of supervisor)

2. Location of the test (city, county, state, name or street location of the line and the stations of the test limits)

3. Date of the test

4. Description of the test section, which must be one type of pipe, of uniform diameter and lining (shell thickness, number of longitudinal seams and whether riveted or welded, spacing of girth seams and whether riveted or welded and types of rivet heads on the inside)

5. Pipe diameter (the method of measurement used, nominal diameter, specified lining thickness, computed net diameter, measured average diameter and number of locations on which the average measurement is based)

6. Description of lining material and method of application (the date of lining if not simultaneous with original installation, application temperature of asphalt or coal tar and the finish of cement mortar, and whether applied by vertical dip, hand daubing or centrifugal spin)

7. History of pipe (dates of installation and test, age at time of test, per cent of age in service prior to test, whether empty or full when out of service, whether out of service intermittently and approximate mean and maximum rates of flow when in normal service)

8. General factors affecting the friction coefficient (standard length between joints, type of treatment given to interior surface at joints to improve flow conditions, number and type of valves and whether situated between reducers, length and diameter of reducers, number of 90-deg. and also of 45-deg. bends, number and degrees of other bends, number of tees and of corporation cocks)

9. Pipe cleaning data (date when last cleaned, method used, and whether lining was applied at this time)

10. Quality of water (average and extreme pH values; ppm. of carbon dioxide, hardness and suspended residue; whether ground water or surface supply; treatment, including chlorination, given to water *upstream* from test section, and whether chlorination improves flow conditions)

11. Methods of conducting tests (length of test section; whether difference in elevation between stations was established by levelling, manometer under conditions of no flow, or alternative methods; method used to measure the volume of flow, whether by Venturi tube, volumetric measurement in tanks or reservoirs, velocity-type meters, pitot tubes, weirs, orifices, or other methods; method of measuring the pressure head, whether by mechanical gage, water manometer, cistern-type manometer, or differential manometer; if the latter, whether mercury, carbon tetrachloride, bromoform or other fluid is used)

12. Data tabulated in the field, on a suggested field data sheet, should accompany the report of the test

13. A summary sheet of test data should also be made, listing in tabular form, for each test, the rate of flow (in gpm. and fps.), the velocity (net pipe area, in fps.), the elevation of the hydraulic grade of both stations (the pressure head not including the velocity head), the friction loss and length of the reach in feet, deducting any head loss through partial obstructions as determined by the differential manometer, and also deducting the length between piezometer taps for such measurements

14. Pipe factor, if pitot tube traverse is made (ratio of mean velocity to center velocity, accompanied by a traverse curve if convenient)

15. Comments of the test supervisor (his estimate of the accuracy of the results; details of the test methods used; photographs of test equipment, of manometer stations, and of recent coupons

from tapping machines to help classify the roughness of the pipe interior).

Conclusion

It is hoped that the emphasis placed on thoroughness will not tend to discourage water works operators from obtaining flow test data. A thorough study of the problem requires a range of data beyond the capacity of the various hydraulic laboratories to supply. While it is not ordinarily feasible for a water works organization to do the precise type of research performed in the hydraulic laboratory, tests made with reasonable care and thorough recording of all the pertinent data will be a valuable contribution, when correlated with other data already available and with that which it is hoped will be available in the future.

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3. Directions for Installation and Operation of the Cole Pitometer. The Pitometer Co., New York.
4. Instructions for Use of Simplex Pitot Tube (or Rod) and Manometer. Simplex Valve and Meter Co., Philadelphia.

Tests on Concrete Water Pipe

By J. L. Irwin

Asst. Engr., Water Pipe Extension, South Dist., Dept. of Public Works, Bureau of Eng., Chicago, Ill.

Presented on May 7, 1946, at the Annual Conference, St. Louis, Mo.

IN July and August 1943 and March 1946 a number of tests were made on a 36-in. prestressed concrete water main placed in service along Rockwell St. in Chicago, between 50th and 67th Sts., on June 25, 1943. This feeder main, supplied from a 48-in. main in 50th St., is 2.125 miles long (1).

Velocity rod meters were used to measure the flow, and pressure gages to observe the pressures, about 500 ft. south of the source of supply and about 200 ft. north of the south end of the concrete pipe (10,668 ft. apart). The pressure gages, which can be read to the nearest 0.25 lb., were checked for accuracy on a dead weight tester before and after each test.

To minimize instrumental and personal errors, rods were marked on one side so the direction in which orifices were placed could be noted. Simultaneous readings were taken with the position of the orifices of the two rods reversed, so that four different combinations were obtained. The rods and gages were then interchanged and the procedure repeated, thus obtaining eight combinations of readings for tests 1–4. The variations caused by the interchange were so slight that the remaining tests were made with four combinations of simultaneous readings. When tests 5 and 9 were made, simultaneous readings were recorded every 10 seconds. During all other tests, the average readings were recorded for each half-minute.

Table 1 shows the number and date of each test, the mean velocity in fps., number of simultaneous readings, friction loss per 1,000 ft., value of C in the Williams-Hazen formula, and value of f in the Darcy formula

$$h = f \frac{L}{D} \frac{V^2}{2g}$$

From 160 to 320 simultaneous readings were made for each of the tests. The results showed the friction loss per 1,000 ft. to vary from 0.29 ft. to 1.74 ft. The values of f ranged from 0.0122 to 0.0132, with a 0.0127 average for the 8 tests. The Williams-Hazen C varied from 144 to 155, averaging 148.

Nine double reverse bends in the section under test take the concrete water pipe underneath other utilities from intersecting cross-streets. These offsets consist of 45-deg. and 22½-deg. cast-iron bends. There are also three 24-in. valves with 36 × 24-in. cast-iron reducers in the line. The losses through these offsets and the reduced sections were neglected; if computed from theoretical formulas and measurements for similar velocities, the average value of f becomes 0.0104, and the average C becomes 166.

TABLE 1

Flow Coefficients in 36-in. Water Mains

Test No.	Date	Mean Velocity *fps.*	Number of Simultaneous Readings	Friction Loss per 1,000 ft. *ft.*	*f*	*C*	Corrected Values*	
							f	*C*
1	7-15-43	2.75	320	0.53	0.0132	147	0.0112	162
2	7-19-43	3.47	320	0.80	0.0129	148	0.0104	166
3	7-20-43	4.24	320	1.18	0.0127	147	0.0103	164
4	7-21-43	5.13	320	1.74	0.0128	144	0.0104	161
5	8- 2-43	2.97	240	0.58	0.0127	151	0.0103	169
6	3-21-46	2.10	160	0.29	0.0127	155	0.0101	176
7	3-28-46	4.545	160	1.31	0.0122	149	0.0099	167
8	3-28-46	4.445	160	1.31	0.0128	146	0.0104	162
Average for 8 tests					0.0127	148	0.0104	166
9	8-10-43	3.91	240	1.30	0.0164	129 (36-in. cast-iron pipe 30 years old)		

* Values corrected for estimated losses through reduced sections and bends.

For comparison, test 9 was made on a 36-in. cast-iron pipe which had been in service for 30 years in Justine St. This test gave an *f* value of 0.0164 and a *C* value of 129.

All values of *f* were computed by arithmetic and checked with a slide rule. Values of C were obtained by the use of logarithms and checked with a loss-of-head slide rule.

Static tests of 100 psi. or more were applied to each half mile of pipe before the ditch was backfilled. Closed tests were made to determine the amount of underground leakage in the 2.125 miles of 36-in. pipe under the normal city pressure of about 40 psi. The results of two such tests, made in July of 1943 and October of 1945, were: 210 gpd., or 2.74 gpd. per inch of diameter per mile of pipe; and 269 gpd., or 3.51 gpd. per inch of diameter per mile of pipe.

Acknowledgment

All tests were made under the direction of J. B. Eddy, Engr., and were approved by B. W. Cullen, Supt., Water Pipe Extension. W. P. O'Neill, Jr. Engr., Water Pipe Extension, South Dist., supervised the Feeder Survey Party which made the tests. The underground leakage tests were made by a party under the supervision of W. P. McElligott, Asst. Engr., Water Pipe Extension, South Dist.

Reference

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Vertical Water Velocity in Deep Wells

By Claud R. Erickson

Mech. Engr., Board of Water and Electric Light Comrs., Lansing, Mich.

Presented on September 19, 1946, at the Michigan Section Meeting, Traverse City, Mich.

LANSING, Mich., is dependent upon deep wells for its water supply. The city now has 43 deep turbine wells, 46 air-lift wells and 3 wells which are now used for observation only. Of this total, 10 were added in 1941 and 12 more completed in 1945. In addition to new drilling, a program of restoration by dynamiting wells of diminishing yield was initiated in the middle of 1943. To date, 14 wells have thus been restored.

All wells are cased off through the glacial drift, which averages 80 ft. in thickness and extends through the Parma formation of sand rock for a total depth of 400 to 450 ft., at which level a thick shale bed almost always appears. The sandstone, which is not always uniform in character, has shale lenses interspersed throughout its depth. Due to this complexity of formation, it is difficult to predict the characteristics of any well. Studies by pumping tests, to determine the permeability of the sandstone, produce such variable results that it can be concluded only that the formation is not homogeneous. Such studies do not indicate at what levels water enters the well, nor the quantity yielded at any given point. To discover these facts, a well caliper and a current meter to measure vertical velocities have been designed and constructed.

Well Caliper

The well caliper (Fig. 1) consists of two spreading arms which actuate the contact on a rheostat. By measuring the electrical resistance, the distance between the arms at any point in the well can be found from a chart based on the relationship between the distance between the arms and the resistance for such distance. In use, the arms are locked in the closed position and the caliper is lowered to the bottom of the well. When the caliper weight strikes the bottom of the well it trips the arm lock, and the arm springs hold the arms against the walls of the well as the caliper is withdrawn.

The caliper thus gives the diameter of the well at any point. Its principal use is to determine the cross-section of the well, so that, when the velocity is measured at that point, the quantity of water flowing can be determined. The caliper can also be used to determine whether a well has been drilled to the full specified diameter throughout its depth and the variation in cross-section after a well has been dynamited. Diameters up to 60 in. can be measured with the caliper illustrated.

Vertical Current Meter

The vertical current meter (Fig. 2) is an adaptation of the stream current meter built for vertical use. It

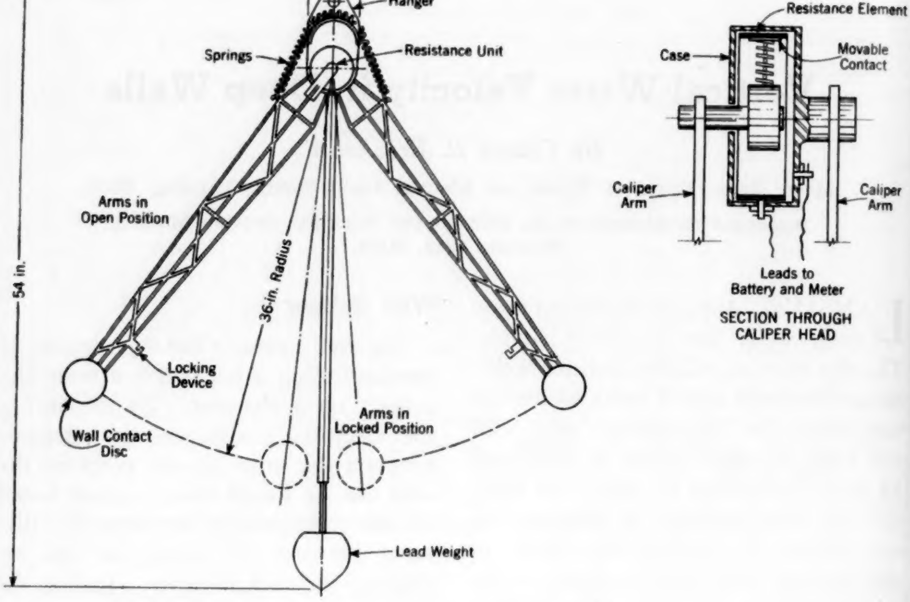

FIG. 1. Well Caliper

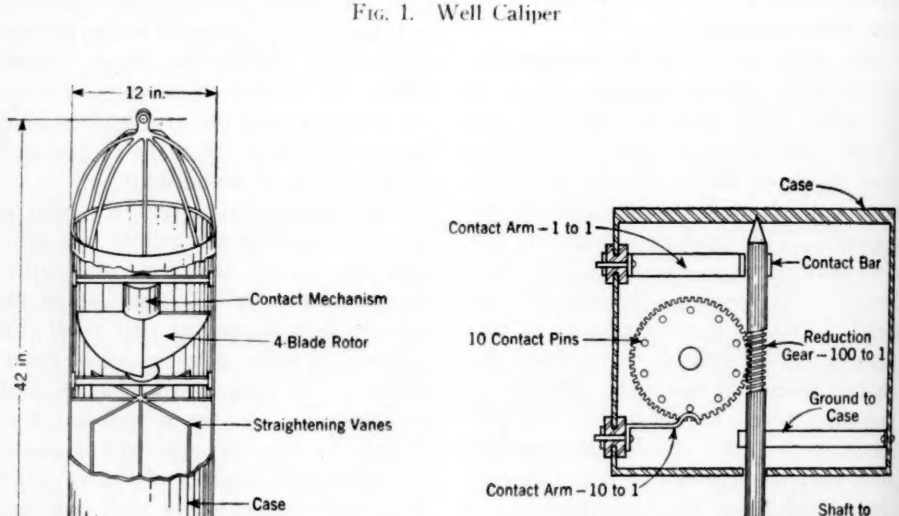

FIG. 2. Flow Meter

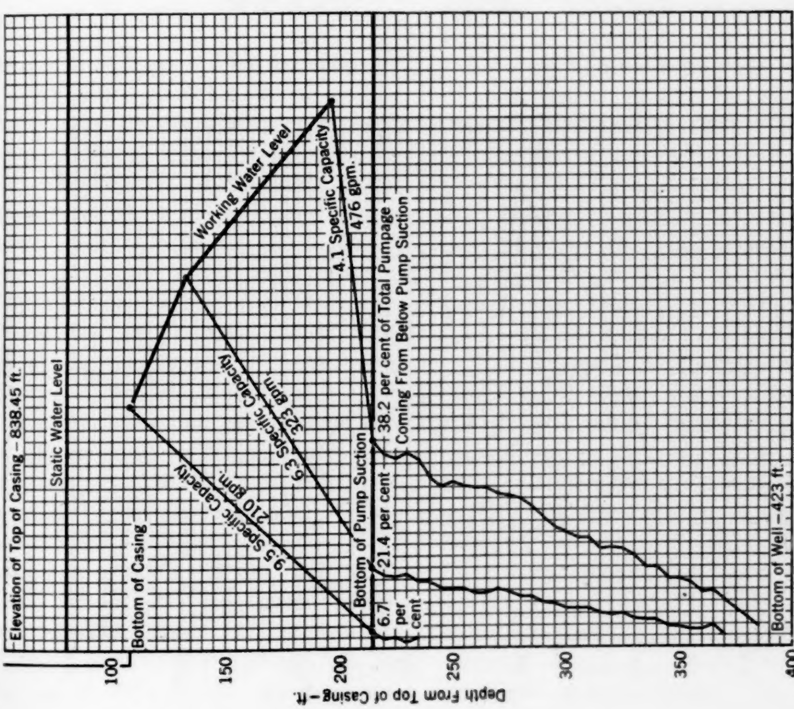

FIG. 3. Study of Well NW-14

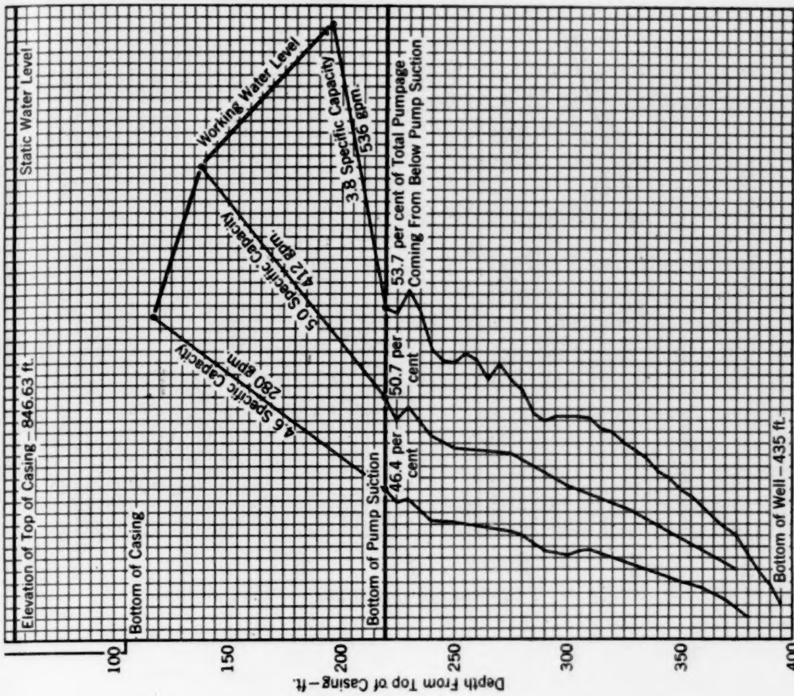

FIG. 4. Study of Well NW-12

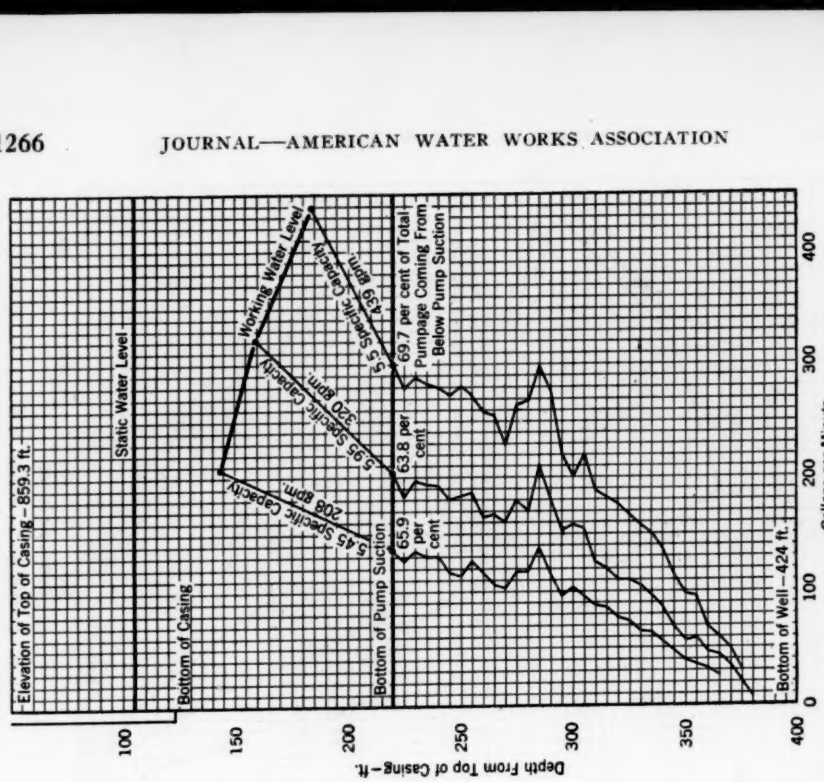

Fig. 6. Study of Well NW-8

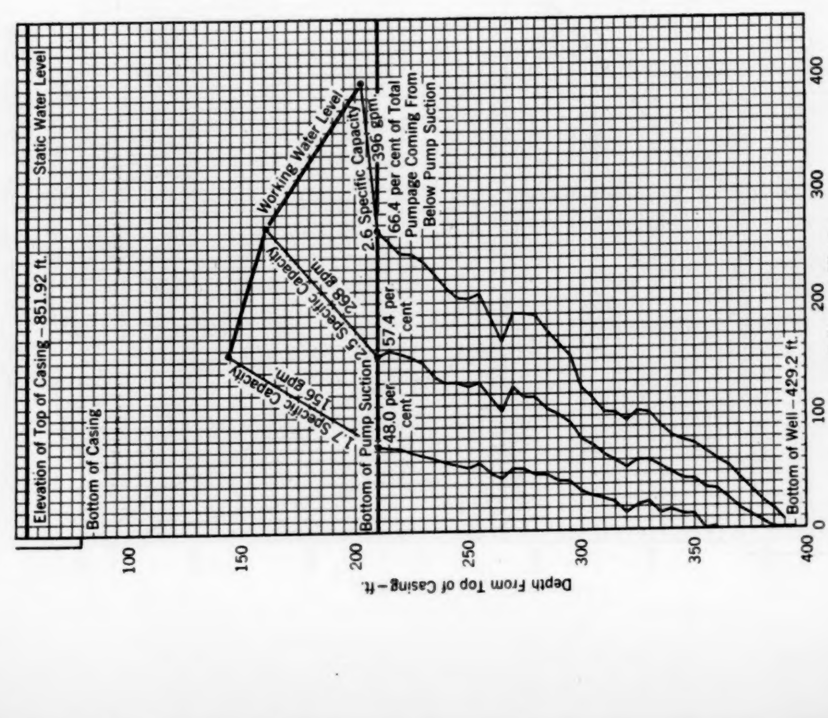

Fig. 5. Study of Well NW-9

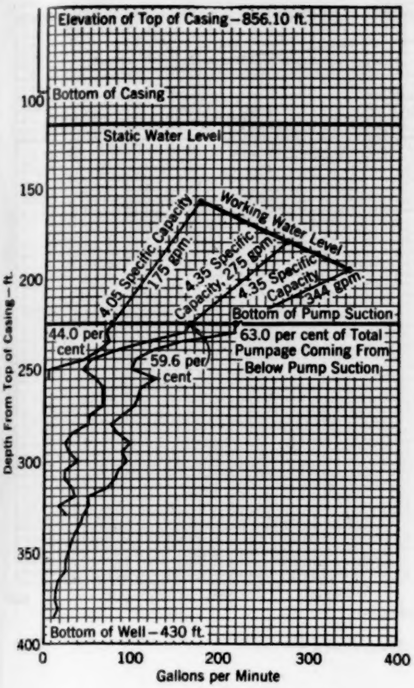

Fig. 7. Study of Well M–8

consists of a sleeve, 12 in. in diameter, with protective guards to house a rotor, contact mechanism and straightening vanes. It is calibrated by being placed in a vertical section of pipe and plotting the revolutions per minute against known rates of flow. The current meter is placed in the well prior to pump installation and then traversed between the well bottom and the bottom of pump suction while pumping at the several predetermined rates. On wells which have just been drilled, the test can be made with economy, as a test pump is generally used, and the flow meter may be reclaimed before the permanent pump is placed.

For low velocities, a record is made of each revolution of the current meter. Built into the meter is a mechanism by which each ten revolutions can be recorded by high flows. Each revolution makes a contact which actuates a magnetic plunger at the top of the well, and a mechanical counter is attached to the magnetic plunger. The plunger and counter are easier to use than the customary earphones.

Well Studies

The results of the caliper and vertical current meter readings for seven different wells were made at the time of test pumping just after the wells were drilled (Figs. 3–9). Three rates of pumpage were used, as near as field conditions would permit to one-third, two-thirds, and the full capacity of the test pump. The figures are arranged in order, with the more uniform results presented first.

Figure 3 gives the data for Well NW–14. The curves below the bottom of the pump suction pipe were established by well caliper and current meter readings. Although the curve is extended to join the point of working water level and the total yield, this part of the curve should be considered merely as one of the limits to which it might be drawn, and is shown only to give the relationship between total yield and working water level. The other limit line for the yield would extend to the elevation of the bottom of the casing rather than to the working water level.

If it is assumed that the distance above the working water level is dry and does not contribute any water to the well, then the cumulative capacity curve would end at the points indicated on Fig. 3. Twice some of the pump column was removed after obtaining current meter data, thereby raising the pump suction pipe. Further tests then indicated that the cumulative curves continue at the same rate of slope to the higher pump suction.

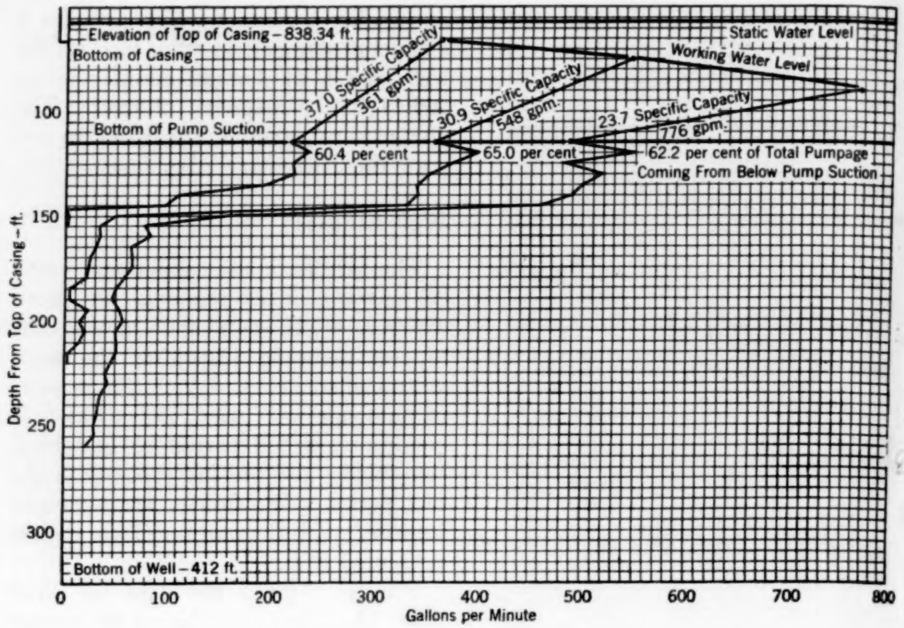

FIG. 8. Study of Well C-18

The water enters the well at a uniform rate for the full depth of the well below the setting, and the volume is proportional to the pumpage rate.

The same relationships hold for Well NW-12 (Fig. 4) as for the previous well. At the higher pumpage rate there is evidence of irregularity in flow, with even an indication that there is a reduction of yield at some points.

Well NW-9 (Fig. 5) is similar to NW-12 (Fig. 3) but with an even more marked loss of flow at the 265-ft. level. It is believed that where the flow slope is reversed, "robber veins" are indicated, and that the well capacity would be improved if such portions were cased. In this well, the initial static water level stood at 54 ft., being drawn down to 203 ft. during full pumpage, but recovering only to 105 ft. Water can be heard falling to the new static level from above.

Well NW-8 (Fig. 6) is located about 1,000 ft. from NW-9 (Fig. 4). Note the occurrence of the same "robber-vein" at the same elevation. Drilling samples show a 15-ft. shale stratum in this vicinity separating two sandstone strata, each over 100 ft. deep.

Well M-8 (Fig. 7) shows a marked deviation from uniform rate of flow into the well. The total yield on this well was low. It has since been dynamited, but test results have not been completed. It would appear that the bulk of the water is coming from the sandstones above the 250-ft. level.

Well C-18 (Fig. 8) and Well C-19 (Fig. 9) show the results from two wells in a higher production area. The specific capacity of C-18, 23.7, is high. Even though the bulk of the water is coming from the higher sandstones, the well for quite some depth contributes to the yield.

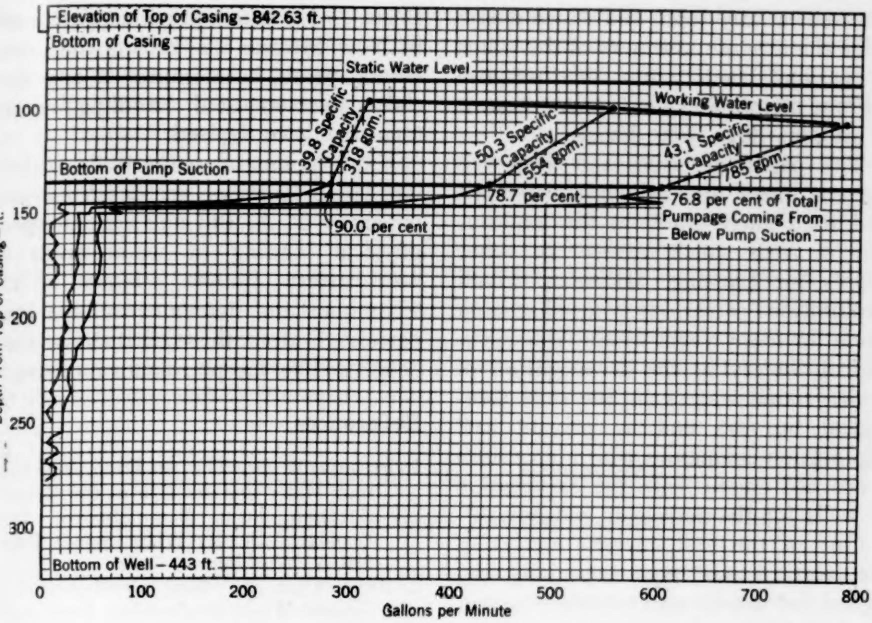

FIG. 9. Study of Well C-19

Well C-19 (Fig. 9) is about 1,000 ft. from C-18 and shows the similarity of this area. Specific capacity is even greater at 43.1 gal. per foot of draw-down. This well was originally tested with the pump suction of 220 ft. but, because of the great amount of water indicated as coming from above the pump suction, the test was repeated with suction at 135 ft.

The data presented were obtained principally to learn more about how water enters a new well. At a later date the tests will be repeated in an effort to learn why there should be a reduction of 70 per cent in the yield over a 5-year period. These data also indicate the depth to which a well should be drilled in any given formation. Perhaps future studies will indicate a maximum entrance velocity of water through the walls of the well that should not be exceeded if the original capacity of the well is to be maintained without decline.

A study of these data also indicates elevations at which a well may best be dynamited to increase its yield.

DISCUSSION—*Albert G. Fiedler*

Div. of Ground Water, U.S. Geological Survey, Washington, D.C.

In order to maintain water wells at maximum efficiency, it is essential that reliable information be available on the location, productivity and change in yield of the respective water-bearing zones encountered by the wells. This information can be obtained by well surveys which measure the diameter of

the wells at any depth and which determine the rate of movement of the water at various levels. For this purpose current meters and instruments known as calipers have been devised.

The paper describes the deep-well current meter and caliper constructed and used successfully in the exploration of wells which furnish the water supply for Lansing, Michigan. Well explorations of various types have been made for many years by the U.S. Geological Survey in the investigation of underground waters, by the state Geological Surveys and by private well exploration companies which have been concerned chiefly with the exploration of wells for the petroleum industry.

One of the first current meters of the vertical-axis type especially designed for use in water wells was that used in 1925 in the Roswell Artesian Basin (1). The equipment first used in New Mexico has, during the intervening years, been improved and special power-operated handling equipment has been devised.

Explorations of water wells, including caliper surveys, are described in a paper by C. A. Bays (2) on the geophysical logging of water wells in northeastern Illinois. Many of the techniques and instruments have been patterned after those that have long been used in the petroleum industry.

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2. Bays, Carl A. New Developments in Ground-Water Exploration. Jour. A.W. W.A., **35**: 911 (1943).

DISCUSSION—*Robert W. Sawyer*

Partner, Malcolm Pirnie, Engrs., New York

The well caliper and the flow meter applied by the author are ingenious and provide the means of acquiring more detailed and accurate knowledge of rock aquifers than has been possible in the past. Tests of rock holes for yields at various depths are usually made by inserting packers to isolate a section of the rock bore and then running a pumping test. This method does not give a continuous traverse and is cumbersome and expensive.

The author is to be commended for his applied engineering which provides an accurate and relatively simple means of determining the economic depth for a well in a field where there are a number of wells drawing from a rock aquifer (but not including the limestones which contain large solution passages).

The graphs should also show the results of the caliper measurements and the text should describe how the meter was calibrated and meter coefficients determined for sections of the bore with nominal diameters larger than 12 in. Such sections could occur where there has been a fall during or subsequent to drilling, or as a result of dynamiting. Where such enlargements occur there will always be some error in computing flows from meter measurements because the caliper only gives the diameter in one plane and does not measure eccentricity with respect to the well axis. Since the lift on the meter is tangent to the rock bore where it passes the pump bowls, it is possible that the meter is inaccurate where such enlargements are longer than the meter itself.

Possibly a meter with a case 10 or 15 ft. long would add to accuracy.

Some of the current meter results show fluctuations that appear inconsistent as the readings approach the pump suction level. This may be due to turbulence, caused by flow from above and below the suction finding its way into the tail pipe.

The clogging which is mentioned may be due to deposition of lime on the walls of the bore or in the sandstone around the bore because the velocity of water in the rock is increased as it reaches the well. Under these conditions, waters high in bicarbonates might deposit calcium carbonate, and it is possible that a good acid treatment with scrubbing and agitation would do as much to restore capacity as dynamiting.

A general plan of the well field or fields in which the wells are located would be helpful in studying and interpreting the results. Such a plan should also contain notes showing which wells were being pumped, pumping level, rate, depth of well and pump setting.

DISCUSSION—*L. J. Alexander*

Design Engr., Southern California Water Co., Los Angeles, Calif.

The ideas suggested by the author are of considerable value to the Southern California Water Co. One of its deep wells, 1,956 ft. deep, is producing several types of water. Various chemical analyses have not told the complete story of the production of this well, and the suggestion of a vertical-flow meter comes as a timely aid. An adaptation of the author's meter was constructed by the R. W. Sparling Co. and will soon be used in tests.

As this particular well is cased to its full depth, the true diameter is known and need not, therefore, be determined. It is hoped that the measurements of current flow will give the correct basic information desired.

In the past, the company has depended for its information about the productivity of various water-bearing formations on the chemical analyses of the water obtained from the formations during drilling operations. It has been practically impossible to obtain a true picture of the waters produced at the various formations by this method. A combination of chemical analysis and measurement of vertical current velocity, however, should certainly give the answers to a difficult problem. The author is to be commended for his work and his valuable paper.

AUTHOR'S CLOSURE

The discussions are all interesting and add materially to the data presented in the article. The equipment which is described and illustrated has been used by the U.S. Geological Survey for determination for vertical velocities in artesian wells, but this is believed to be the first application to wells in which deep-well turbopumps are necessary to lift the water to the surface.

Due to the relatively low vertical velocities in the wells tested, any error incurred because the meter is near the

well wall instead of in the center will be slight. The meter area covers a large part of the cross-sectional area of the well.

The criticism that the calipers measure only on one plane is well-founded. There is now under construction a calipers that will measure in two planes; having four arms instead of two. When measurements are made in one plane only, some of the error can be eliminated by making several trips up the well.

All measurements were made in wells having a nominal diameter of 13¼ in. All wells were drilled with cable tools.

After these wells have been in operation for a few years, duplicate data will be obtained to compare with the original findings. It is hoped that a limiting velocity for water entering the well can be determined which will prevent clogging. The same principles that are used to design screens for wells may be proper to apply to rock wells when backed by sufficient data.

Municipal Water Works Organization

Committee Report

Wendell R. LaDue, Chief Engr. & Supt., Bureau of Water and Sewerage, Akron, Ohio; Chairman, A.W.W.A. Committee on Municipal Water Works Organization

Presented on May 7, 1946, at the Annual Conference, St. Louis, Mo.

THE need for a study of top management in water works and the determination of universal good practice in this most essential phase of water works endeavor was very forcibly brought to the attention of the Association by Past-President Louis R. Howson in a paper presented before the Indiana Section in April 1941 (1). He continued his efforts by inviting the attention of the Board of Directors to the problem with the result that the Board authorized the formation of the Committee on Municipal Water Works Organization at the Toronto Convention in June 1941.

Upon Past-President Howson's invitation, a survey committee was selected, composed of members of the Association (with the author as chairman) known to be especially interested in the practical problems of carrying out good management of municipal water utilities and representing all sections of the United States and Canada. This committee was later expanded to 25 members, a rather large group, but by no means too large for the problems confronting it (See Table 1).

During the past five years changes in personnel have inevitably occurred due to the vicissitudes of war, death, withdrawal and change of professional affiliations. The magnitude of the problem was not at first generally appreciated, but, as the widespread and varied background of the basic factors emerged, it became advisable to assign from one to four states to each committeeman for detailed study by special assignment. He, in turn, was free to solicit aid from interested water works men.

To co-ordinate the work of the committeemen, the country was divided into three regional committees:

1. *Pacific.* Comprising 15 states under the chairmanship of J. S. Longwell, Chief Engr. and Gen. Mgr., East Bay Municipal Utility Dist., Oakland, Calif.

2. *Central.* Comprising 17 states under the chairmanship of D. L. Erickson, City Engr. and Director of Parks, Public Property and Improvements, Lincoln, Neb.

3. *Atlantic.* Comprising 16 states, the District of Columbia and Canada under the chairmanship of Charles E. Moore, Mgr., Water Dept., Roanoke, Va.

As could be expected, the committee's activities were very much affected and curtailed by the war and were not pushed as vigorously as would have been possible in peacetime. The burdens of the war effort fell heavily upon many of the committee's personnel in the centers of the war's industrial activities. Water was an essential war product and as such demanded the

TABLE 1

Personnel of Committee on Municipal Water Works Organization, March, 1946

Dist. No.	Political Subdivision	Name	Title	Organization	City	State
	Chairman	W. R. LaDue	Chief Engr. & Supt.	Bureau Water & Sewerage	Akron	Ohio
1	Maine, New Hampshire, Vermont	H. L. Clark	Supt.	Sanford Water Dist.	Sanford	Me.
2	Massachusetts, Connecticut, Rhode Island	W. P. Melley	Supt.	Water Dept.	Milton	Mass.
3	New York, New Jersey, Pennsylvania	R. M. Roper	Engr. & Gen. Mgr.	Board of Water Comrs.	East Orange	N.J.
4	Virginia, Maryland, Delaware, Dist. of Columbia	C. E. Moore*	Mgr.	Water Dept.	Roanoke	Va.
5	North Carolina, South Carolina, Georgia, Florida	A. K. Maddox	Field Repr.	Loan Agency of RFC 2905 Peachtree Road	Atlanta	Ga.
6	Ohio, West Virginia	E. E. Smith	Gen. Supt.	Dept. Water & Sewerage Treatment	Lima	Ohio
7	Michigan	O. E. Eckert	Gen. Mgr.	Board of Water & Electric Light Comrs.	Lansing	Mich.
8	Kentucky, Tennessee	L. G. Lenhardt	Gen. Mgr.	Board of Water Comrs.	Detroit	Mich.
9	Illinois, Indiana	W. R. Gelston	Supt.	Water Works Com.	Quincy	Ill.
10	Alabama, Mississippi	G. L. Fugate	Chief Engr.	Water Dept.	Houston	Tex.
11	Wisconsin	L. A. Smith	Supt.	Water Dept.	Madison	Wis.
12	Minnesota	W. A. Peirce	Mgr.	Racine Water Dept.	Racine	Wis.
13	Iowa	D. L. Maffitt	Gen. Mgr.	Des Moines Water Works	Des Moines	Iowa
14	Missouri, Kansas	M. P. Hatcher	Supt.	Dept. of Water	Kansas City	Mo.
15	Arkansas, Louisiana	G. J. Rohan	Pres.	The Rohan Co., Inc.	Waco	Tex.
16	North Dakota, South Dakota	H. H. Brown	Supt of Water Works	Dept. of Public Works	Milwaukee	Wis.
17	Nebraska	D. L. Erickson*	Director	Parks, Public Property & Improvements	Lincoln	Neb.
18	Texas, Oklahoma	H. A. Hunter	City Mgr.		Lubbock	Tex.
19	Colorado, Wyoming, Utah	G. F. Hughes	Mgr.	Board of Water Comrs.	Denver	Colo.
20	Arizona, New Mexico	B. S. Grant	Asst. Chief Engr. of Water Works	Dept. of Water & Power	Los Angeles	Calif.
21	Washington, Montana	M. W. Plummer	Asst. Mgr.	Butte Water Co.	Butte	Mont.
22	Oregon, Idaho	M. H. McGuire	Mgr.	Water & Light Dept.	McMinnville	Ore.
23	California, Nevada	J. S. Longwell*	Chief Engr. & Gen. Mgr.	East Bay Municipal Utility Dist.	P.O. Box 720 Oakland	Calif.
24	Canada	J. S. Keith	Gen. Mgr.	Windsor Utilities Com.	Windsor	Ont.

* Regional Chairman: Atlantic—Dists. 1–5, 24; Central—Dists. 6–14, 16, 17; Pacific—Dists. 15, 18–23.

all-out efforts of water works management. It is indeed gratifying that many carried on despite these circumstances, as their willingness to assume the additional load is clearly indicative of their *personal* interest in and understanding of the problem. Renewed effort on the part of all committeemen is now essential to obtain early and increased returns from the committee's findings.

Purpose of Committee

The committee, believing that improvement is always possible, has as its goal the improvement of management of municipal water works systems.

Good management results from continuity of policies on expansion of both facilities and operations, from sound financing, efficient administration and good service. It is as essential for small plants as for large water works systems; it will become mandatory as the public realizes its importance. All too often we think of management as confined quite narrowly to more or less routine operating functions. In the more practical analysis, its functions must include sound planning for the future and the co-ordination of needs, plans and accomplishments with the financial obligation and ability to support them.

Good management requires, further, the development of a vital routine which will carry forward an orderly program with adequate assurance that future developments will largely be provided for as they arise. Also, good management frequently includes thinking, planning and even building by the present generation to provide for some of the requirements of those who are to follow. Water works management, therefore, is a specialized profession requiring experience in a most difficult and essential field.

Discussion of Management

There is probably no topic which receives more discussion among water works men than that which might be properly classified under the general heading of "management."

For purposes of discussion, let us state that the water works profession will accomplish much to improve its own status if it can develop better top management. This is the subject that is really being discussed when titles read: "Factors Affecting the Continuity of Employment," "The Diversion of Water Works Revenues," "Improving the Status of Water Works Operators," "Revenue versus General Obligation Bonds," and, keeping very much abreast of the times, "Public Relations," "Personnel Problems," "Unions and the Municipal Employee" and "Do Water Works Employees Have the Right To Strike?"

Is it not possible that these specific problems result from the inadequate attention that is given to the underlying, fundamental reasons for their occurrence? In other words, do we not spend our time discussing the *results* of bad management, whereas we ought to be doing something about analyzing the *causes,* diagnosing the resulting weaknesses and then taking the steps necessary to correct them? With such an impractical multiplicity of state laws, local ordinances and charters, is this truly not a challenging problem?

At the outset we find ourselves becoming quite philosophic and altruistic in realizing and appreciating the other fellow's position. The ramifications are many but the generalities few. There are many cities in which the water works management is directly under the city council, the personnel of which is continually shifting. With such an organization, centralized top responsibility is generally slight, since there is no real concern about long-range planning for the work of later administrations. There are very often too many potential "bosses," and the superintendent, no matter how good, may be subject to change with the city administration.

Under the city-manager form of government, water works management is only as good as the manager, who, in turn, is subject to the changes and whims of a council or other elected board of similar nature. Changes in city managers are such that his appointees usually pass with him.

A third general form of top management is that of the special water works commission, usually consisting of three to five men with overlapping terms of such length that no one political administration can upset the board by a majority of new appointments. Such a board is responsible to no one but the public which it serves. It is nonpolitical, its continuity is assured and it attracts to its membership the best qualified men of the community. This form of management lends itself to a continuity of employment of water works personnel, the maintenance of high efficiency, the establishment of

rate stability, continuing policies of construction and operation, and the furnishing of a maximum of satisfying service at a minimum of cost.

In all, we are brought face to face with one of the most baffling challenges of human nature and behavior: that people get and do what they want, good or bad. When a superintendent in an ideal, commission-operated water works asks why a certain city countenances the shortcomings of a council-controlled operation, the only answer forthcoming is: "Because people want and like it that way."

For this very reason, regardless of its logic, a uniform type of top management is not always possible or perhaps even desirable. Past history and local conditions will weigh heavily and must be considered. The comprehensive study of the practicable alternatives in municipal water works management and organization, and the development of somewhat ideal types of water works administrative control, which with variations could be adapted to practically all conditions, has been and will continue to be a most constructive undertaking of the committee. When typical ideal forms are made available by the Association, they will be helpful guides for those interested in improving water works management. This committee's findings would thus be instrumental in improving the status of water works employees and the conditions under which they are employed.

Forms of Management

The recognized forms of public water utility management quite naturally vary depending upon size and extent—volume and territory—as follows:

1. *Metropolitan Districts (including several municipalities)*. This is a state subdivision, topping municipalities.

2. *Water Boards (organized within a single municipality)*. Here the water utility control is independent of the general city government.

3. *Municipal Departmental Organization.*

(1) City Manager
(2) Charter City
(3) Mayor and Council
(4) Service Director

4. *Small City and Village One-Man Organization.* This is a simplified form of item 3.

Although the investigations of the committee have been confined to the water works field, the other public utilities—sewage treatment works, garbage and refuse plants, gas and electric plants—must be considered insofar as they may be connected with the water utility through over-all management and commercial activities. Of late there has been a decided trend toward the amalgamation of these municipal utilities under one operating, managing head. Thus the water works manager is called upon to exercise his experience in a somewhat foreign field. There are many notable instances of water works men providing outstanding examples of leadership. This speaks well for water works managers with progressive outlooks.

Procedure

As the committee wished to obtain a cross-section of the background and thoughts of water works men, a questionnaire was developed for general distribution. Since the spring of 1942 copies have been sent to water works organizations in each state or province, selected in such a way that a true cross-section of the various forms of organizations would be obtained.

This questionnaire requested information on:

1. Name of municipality.
2. Population and area.
3. Water consumption, services, mains, hydrants, investment, income.
4. Form of municipal government.
5. Form of water works management.
6. Control of water works by municipal organization.
7. Funds (collection and distribution).
8. Extent of state supervision.
9. Review of state laws permitting or preventing formation of metropolitan districts.
10. Review of state laws permitting or preventing formation of a separate water board within a municipality.
11. Description of board or district organization.
12. Digest of laws governing management of water works systems.
13. Other utilities and functions municipally owned and combined with that of water works management.
14. Comments upon the existing form of management and suggested changes.

The questionnaires enabled the committeemen to contact managers of plants, obtain a somewhat long-distance insight into their managerial problems, note what limitations existing legislation imposed on them and what hope there was for constructive criticism of things as they were. A surprising number of men were reticent on this latter issue.

Questionnaire Results

Of 590 questionnaires sent out, 255, or 44 per cent, have been returned to date. Many more are expected. A present summary of replies indicates a general expression of confidence in the board or commission form of management for water works systems. Of those answering, 93, or 36 per cent, operate the water utility under a separate board or under a commission form of government. Often one of the commissioners is in charge of the utility.

There were, however, many expressions of confidence in, and many successful examples of, other forms of management for water supply systems, large and small. Some managers expressed direct aversion to change to *any* other system, no matter how good it was elsewhere. It was quite obvious that many water works managers were not much concerned with the changing of *any* form of management in which they were not having trouble locally. Often they had set up an arrangement which satisfied all concerned, although it may have been in the "twilight" zone of legality, with regard to local or state statutes. An outstanding example was the satisfaction in a system which permitted the diversion of *all* surpluses in the water fund to *any* city operation. This is a far cry from the law that usually exists permitting *no* diversions whatsoever. The inference is that if a system works and everybody is happy, let it alone. Are water works men, as operators themselves, to quarrel with that philosophy? Can any one person answer that question for all?

Committee Papers

At the Cleveland Conference in June, 1943, L. A. Smith, a member of the committee, and William Storrie presented papers on commission-controlled water works operation in Wisconsin and Ontario respectively (2, 3). The reception of these papers by those present and a study of them indicated that a very desirable form for summarizing and presenting certain committee data had been developed. Consequently, these papers were abstracted and sent to the committeemen for their guidance. At Milwaukee in 1944, Moore, Erickson and Longwell, Regional Chairmen of the committee, presented papers on operation of water works utilities in

Virginia, Nebraska and California, respectively (4–6). The operation of Maryland utilities was described by H. B. Shaw, acting under Moore (7). Likewise, the operation under Canadian conditions has been further set forth by Berry, Macnab and Scott (8–10). Under the guidance of Committeeman Roper, a digest of conditions in New York was prepared by Rowe and Engelder (11).

Condensed data from ten other states are in the hands of the committee at present and scheduled for publication as evidence of committee activity.

Future Effort

The foregoing gives a general background of the present status of the committee's work and thought. As previously indicated, the work was a war casualty. Now that the war is over, a vigorous and active resumption of the committee's activity contemplates two phases of effort:

First, the resumption of the follow-up program, established after the return of the 1942 questionnaires, embracing the following:

1. An intensive effort to obtain additional and more complete returns to questionnaires, especially those missing from smaller municipalities of known special interest.

2. Closer study of returns and more direct contact with those who seem to have worthwhile ideas, with the intent of developing them.

3. Direct contact with representatives in capital cities for submission of digests of the state and municipal laws governing water works management, and consideration of the revisions required in these codes to obtain desired results.

4. Obtaining copies of typical and eminently successful local ordinances for examination and possible establishment as models.

Next, a continuation of the preparation of papers covering operations in the various states and provinces. These summarized statements prepared by members located in the given states have been and will continue to be of great aid to the various committeemen.

With these results and data before the committee, it has been and is the intention of the committee to: (1) bring out and correlate the essentials of all types of organization, weighing the favorable features; (2) set up generalized model forms applicable to *each* type; and (3) recommend and suggest a summarization of the enabling legislation in each state, directing attention to the study by the individual of the limiting features as they digress from the model form.

An important result of the studies thus far has been the conclusion that the first and foremost requirement for good management is a set of sound basic laws of the political division under which the utility operates. Essential laws may be obtained only by careful study of the existing limiting legislation, along with an effort by local managers of utilities to favor and promote, through appropriate legislative organizations, the necessary laws to permit the form of management deemed best for the utility.

In the consideration of the best form for the utility, the provocative question has been raised: "Is the water works manager unselfish enough to judge the best interests of the city?" The form best suited for the management may not be best for the community as a whole. It might be entirely proper and legal to divert funds of the water works utility for use of other city functions. This theory is especially perti

nent in the use or misuse of water works funds. If what happens is distinctly understood by all, and the utility law provides adequate protection and maintenance of the system by adequate financing, it may be satisfactory and expedient to divert excess funds to other city functions. This is being done in many systems without opposition.

Although a study of results and numerous discussions indicate a general feeling that the board form of management best protects the utility, there is a reluctance to disturb existing forms of management, especially where they apply to a large number of small organizations. All concerned must not lose sight of the needs of the water works men in the small municipalities and in one-man organizations, who must be protected from their "friends" by means of adequate basic laws. These men may well look to the Association for aid, advice and comfort.

The committee has a great deal of public relations work to do with the individual utility manager. There appears to have developed an undercurrent idea that the committee (and the Association) is endeavoring to develop a regimentation of organizations. This is not at all the intent of the committee, which is endeavoring to pick out and develop *several* forms suitable for the various conditions to be met and to indicate and provide means for personal comparison. Solving organizational problems must be done at the local level. The Association can only provide the incentive.

State comparisons have exerted a stimulating effect upon committeemen, and a desire for more effective comparison has arisen between the various states and their water works employees. One of the great values of comparison is the feeling of satisfaction that comes from solving one's own problems in the light of the difficulties others have experienced. Others have worries, too, and that thought of mutuality alone seems to be a great morale builder.

A pre-eminent corollary of this committee's functions will be the work of the new Committee on Public and Personnel Relations. These relations are definitely problems of management.

It is submitted that water works executives are confronted by a "king-size" problem, having an all-important function in the water works field and in the national social development. It is not a problem to be considered hurriedly but one that demands careful thought and study of local history and geographical distribution. Its ramifications affect every phase of life of the water works man in his chosen field.

Perhaps it will be found that the problem of management is really that of management personnel. Are individuals being developed who have the necessary qualities of leadership? What is being offered to young men entering the profession besides hard work under adverse conditions, minimum compensation and scarcity of help? In spite of these discouragements, outstanding individuals have been and are being developed, who rise above the circumscription of disheartening local conditions. It has been said that "Few men have the courage to appear as good as they really are." We do have that outstanding few; let us develop more!

Suggested Legislation

In order that committees and those members of the Association desiring to take an active part in this problem may be directed, the following outlines of possible local legislation have been prepared for the alternate conditions of an

independent board or a department of the local political organization. It must be recognized that these are outlines only which must be modified to meet the limiting conditions set by local, state or provincial legislation. If deemed advisable, these restrictions might be modified by concerted action on the part of those interested in good management of municipally operated water works utilities.

Independent Water Board or Commission

Board

The Water Board shall consist of three electors of the city, and no more than two shall be registered voters of the same political party.

There shall be created a nominating committee for the purpose of nominating Water Board Commissioners. The nominating committee shall consist of the head of the school system, the head of the judicial system and the head of the local business organization.

The nominating committee shall nominate three candidates for each vacancy. No more than two of the nominees shall be registered voters of the same political party.

From this list of nominees the chief executive with the consent of the legislative body shall appoint the commissioner.

The term of each commissioner shall be six years, one commissioner being appointed every second year. Commissioners having served a full six-year term shall be ineligible for reappointment until after a lapse of two years. A commissioner shall not be an employee of the city nor a former employee with less than one year separation, nor shall he become an employee of the city during his term of office as Water Commissioner. If he does so, his office as Water Commissioner shall be declared vacant.

The chief executive shall notify the nominating committee of existing or future vacancies. Upon failure of the nominating committee to act within 90 days after being notified by the Mayor that a vacancy exists, or after termination of a commissioner's term of office, the chief executive shall appoint a commissioner of his own choice.

Commissioners shall serve without compensation other than out-of-pocket expenses connected with the work of the Water Board.

The chief executive shall have the power, by and with the consent of the legislative body, to remove any commissioners for inefficiency, neglect of duty or malfeasance in office after public fact-finding before the nominating committee.

Organization

Immediately upon appointment, the commissioners shall meet and elect a Chairman and Treasurer, who shall each hold office for a period of two years. The Board shall hold monthly meetings, which shall be open to the public, not later than the 15th day of each month.

Powers

The Water Board shall have all of the powers which are now conferred by law and the state constitution upon city officials for the management and operation of municipally owned water utilities.

The Water Board shall have full legislative power in all matters concerning the management and operation of the water utility. Full administra-

tive and executive powers shall be vested in a Chief Engineer and General Manager.

The Board shall have the power to make such by-laws and regulations as it deems necessary for the safe, economical and efficient operation and management of the water utility and the protection of the water works property. Such by-laws and regulations shall have the force of ordinances when not repugnant thereto or to the constitution or laws of the state.

The Board shall have the power to co-operate with the proper health departments for the sanitary protection of the waters and water works system.

The Board shall have the power to provide by regulation for punishment of violations of the by-laws and regulations enacted by it. Such penalty may be either fine or imprisonment, or both, as may be determined by the Board.

The Board shall have the power to enter into contracts for the purpose of constructing, extending, maintaining and operating the water works and to enter into contracts for the sale of water.

Chief Engineer and General Manager

Subject to the exclusive direction and control of the Water Board, the Chief Engineer and General Manager shall have full administrative and executive power to manage and operate the water works, having charge of the construction, extension, maintenance and operation of the water works. He shall also be the Secretary of the Board.

The Chief Engineer and General Manager shall be appointed by the Board. He shall hold his position subject to the Board's action only, but can be removed for cause after a public hearing before the civil service commission of the city acting as a fact-finding board.

The Chief Engineer and General Manager shall be a registered professional engineer. He shall be a person experienced in the operation of a water works of comparable size. Upon his appointment, he need not be a resident of the given state or political subdivision.

Employees

Provisions of the city charter to the contrary notwithstanding, all appointments and promotions in the Water Board shall be made by the commissioners according to merit and fitness, to be ascertained, as far as practicable, by competitive examinations.

The Board shall make rules and regulations for the enforcement of this provision and shall establish seniority provisions.

Provisions of the city charter to the contrary notwithstanding, salaries or compensation of the employees of the Board shall be in accordance with the prevailing rates of salary and compensation for services rendered under similar conditions of employment and of vacation, sick leave and retirement privileges for like employment in the industry, generally and without reference to other departments of the city.

Purchases

Notwithstanding any provisions of the city charter to the contrary, purchases for the Board shall be under control of the Board, which shall authorize all contracts in excess of $5,-000. All contracts in behalf of the Board shall be made by the Chairman of the Board, upon approval of the Board, after competitive bidding fol-

lowing due advertising in accordance with the law. The Board shall have the right to award contracts on the basis of the bid it deems best for the water works. In extreme emergency, the Board may award contracts on the basis of competitive bidding without advertising.

Notwithstanding any provision of the city charter to the contrary, purchases for the Board shall be made by the Chief Engineer and General Manager acting as purchasing agent.

Finances

Within 10 days after the organization of the Water Board, the Treasurer shall set up a system of accounts for the water works. Upon notification by the Treasurer that the accounts have been set up, the city fiscal officer shall furnish the Treasurer with a balance sheet of the water works accounts with the supporting data, and shall transfer to the Treasurer all funds and investments of the water works in his possession.

The funds derived from the operation of the water works or from the sale of bonds for water works purposes shall be expended only upon authorization of the Treasurer. Such funds shall be used for the construction, extension, maintenance and operation of the water works utility only, including capital and interest payments on bonds issued therefor.

Bonds

The Board may, if in its judgment it seems best, issue bonds for the construction or extension to the water works system. Bonds so issued may be either general mortgage or revenue bonds, and all requirements of the state shall be complied with in their issuance.

Budget and Records

The Chief Engineer and General Manager shall prepare and submit to the Board at its November meeting an annual budget of income and expenditure for the ensuing year, this budget to be open for inspection until its adoption at the December meeting.

At the January meeting, the Chief Engineer and General Manager shall submit a report of the activities of the preceding year. Copies of the annual report and budget shall be submitted to the city administration for its files. All records shall be kept in accordance with the requirements of the state supervisory department responsible for such records. Records of the Board are considered public property and are open for inspection in accordance with the state law governing this matter.

Date

The Board shall assume operation and management of the water works on (a given date).

Pending the transfer of funds provided by the section on "Finances," the city fiscal offices shall be responsible for the funds of the Board and shall sign all checks for payment from the water works fund.

City Water Department

Department

There is hereby created within the city government a Department of Water for the operation and management of the water works system.

Chief Engineer and General Manager

Subject to direction and control of the chief executive, the Chief Engineer and General Manager shall have full administration and executive power to manage and operate the water works

having charge of the construction, extension, maintenance and operation of the water works, all as provided by the state law for such operation and management. He shall co-operate with the head of the Department of Health in providing for adequate sanitary protection of the waters and water works system.

The chief executive shall appoint the Chief Engineer and General Manager. Such appointment shall be in accordance with the provisions of the civil service rules and regulations except that, notwithstanding any conflicting provisions of the rules, the Chief Engineer and General Manager shall be a registered professional engineer and surveyor, shall have had experience in the operation and management of a water utility of comparable size and, when appointed, need not be a resident of the city or the state in which the city is located.

Employees

Subject to the rules and regulations of the Civil Service Commission, the appointment, transfer, promotion and separation of all employees of the department shall be made by the Chief Engineer and the General Manager.

Funds

The funds derived from the operation of the water utilities, including the proceeds from the sale of bonds for water works construction and extensions, shall be kept in a separate fund and shall be disbursed by the city fiscal officer only upon authorization of the Chief Engineer and General Manager.

Funds derived from the operation of the water works or from the sale of bonds for water works purposes shall be used for the construction, extension, maintenance and operation of the water works utility only, including capital and interest payments on bonds issued therefor.

Contracts and Purchases

Provisions of the city charter to the contrary notwithstanding, all contracts for the water works utility shall be entered into by the Chief Engineer and General Manager; likewise purchases of all equipment and materials shall be made by him acting as purchasing agent and in accordance with specifications prepared under his direction.

Budgets and Records

The Chief Engineer and General Manager shall submit to the chief executive, not later than November 1, an estimate of revenue and expenditure for the ensuing year. If this budget is not approved by January 1, the Chief Engineer and General Manager shall operate under the terms of said budget notwithstanding provisions of the city charter to the contrary.

All records of the department shall be kept in accordance with the requirements of the state supervisory agency.

An annual report shall be prepared and submitted to the chief executive not later than March 1 of the following year.

Acknowledgments

All water works men interested in good management—and who is not?—are asked to give this subject careful thought and send to the committee ideas which are thought worth while.

The work of the committee is becoming known among water works men. Many inquiries have come to the Association's headquarters and to the committee asking for advice about sug-

gested forms of management and other phases of the management problem.

Thanks are extended to the committee and the other members of the Association who have worked with the committeemen in gathering and assembling essential and necessary data. Special mention should be made of the Regional Chairmen—Moore, Erickson and Longwell—for their work in directing and correlating the work of the committeemen.

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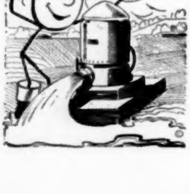

Control of Oil Lines in Watershed Areas

An Editorial Review by Daniel N. Fischel

Assoc. Editor, Journal of the American Water Works Assn., New York

FOR several years, water works executives have been disturbed by the installation of oil or gasoline pipelines across watershed areas. Generally the larger portions of these areas are not owned by the water utilities, which are empowered to use them by the right of eminent domain. The legal right to prevent oil-line installations in such circumstances seems not to have been definitely established.

These pipelines are a threat to public water supplies and it is time they were recognized and fought as such. Records from only a few areas indicate that breaks in petroleum products pipelines on watershed areas have occurred with disturbing frequency.

Breaks in Oil Lines

On four occasions in Illinois—at Centralia, Pana, Vandalia and Norris City—breaks occurred and contaminated the impounding reservoirs with floating oil, requiring considerable effort on the part of water works men to prevent the spread of the contamination into the distribution system. At Centralia, straw had to be spread over the surface of the reservoir and burned along with the oil it had absorbed. At Norris City, it was a feed line to a storage tank which broke, spilling oil into a ditch that drained into the reservoir. The break at Vandalia covered the supplying river with a heavy layer of oil. Only the exercise of extreme caution prevented contamination, as the water is pumped directly from the low-lift station into the plant.

Experience in other states seems to have been similar, especially with the older pipelines. Often defective material, poor workmanship or hasty installation has been responsible, but it must be recalled that such breaks mean considerable expense and disruption of service to the oil-line companies, which presumably do all in their power to minimize them. The reputed total of over 100 breaks which have occurred in two government-installed pipelines, used during the war years to move petroleum products from the producing areas of the southwest to the eastern seaboard, indicates that even when use is made of the best in materials and knowledge, and when unlimited funds are available, breaks in pipelines do occur.

Nor are all "breaks" accidental. A planned discharge of gasoline near Batesville, Ind., very seriously endangered the reservoir of the Batesville Water Works and indirectly threatened the downstream supplies of Osgood and Versailles, Ind. Apparently a gasoline pipeline was being cleaned by a cylindrical cutter propelled by

the fluid behind it, and the line was deliberately opened to permit the removal of the cutter at the Batesville oil-line pumping station, the gasoline pouring into a prepared pit. The amount of gasoline that had to be released before the cutter would emerge seems to have been miscalculated, however, so that from 100,000 to 200,000 gal. of gasoline overflowed the pit and spread over the countryside. Perhaps 25,000 to 40,000 gal. finally flowed into a dry creek bed which led directly to the reservoir.

The first two of a series of dams built to check the gasoline did not hold, and, at the time a Board of Health investigator arrived, only the last was holding back the flood. Even when this dam was being weakened by melting snow, a proposal by the water works superintendent to fire the gasoline had been opposed by the gasoline pumping station attendants, who feared that the flames might spread back to the station. They also minimized the danger by arguing that gasoline floating on top of the water would not mix enough to cause taste difficulties and would overflow later on and so be eliminated.

The Board of Health investigator, however, pointed out that the volatility of gasoline would certainly cause taste and odor difficulties even if contamination were only minor, and only heavy carbon treatment, followed by good coagulation and free residual chlorination, might remove the taste. Although the oil company did co-operate by building a fourth dam and eventually pumped the gasoline out of the stream bed without contaminating the reservoir, it should be noted that the lack of clearly defined, contractual obligation on their part jeopardized the water supply more than was necessary.

Hazards of Petroleum Pollution

Even the methods for disposing of oil before pollution can take place—the most common of which is burning—must be considered hazardous. The great majority of impounding reservoirs are surrounded by forest cover, with plantings in evergreens common. The menace of fire to these trees and, indirectly, to the water supplies which the trees are intended to protect, is a constant problem for watershed patrols.

On the other hand, failure to remove the oil is even more unthinkable. It can hardly be denied that the results of even mild oil pollution are very troublesome. Tests submitted to the laboratories of the Ohio Dept. of Health, which was consulted by the Mahoning Valley Sanitary Dist. when the Sun Oil Line Co. proposed to construct a gasoline pipeline across its watershed, showed that 0.004 ml. of gasoline, when shaken in a liter of Columbus tap water, produced "a strong disagreeable odor characteristic of gasoline." A doubtful taste was produced by 0.002 ml., and no taste was produced by 0.001 ml. The tests further showed that when 0.2 ppm. of chlorine was present in the tap water, only 0.002 ml. of gasoline produced the strong taste.

Furthermore, more widespread pollution can be quite serious. The experiences of H. Cable Cramer, manager of the Catlettsburg, Kenova & Ceredo Water Co., of Catlettsburg, Ky., which has its river water intake three miles downstream from an oil refinery, should prove this. In 1944 the refinery lost several thousand gallons of naphtha into the river, and for some days the entire supply, even after intensified treatment and filtration, was "unfit for household use or drinking."

Some people even questioned its ability to extinguish fires. In the same year, the disposal of refinery wastes into the river imparted a heavy gasoline taste to the water which made it quite objectionable for a period of several days.

At another place, in the early part of 1944, a series of seven leaks and breaks occurred in the region near Goshen and Topeka, Ind., in the pipeline connecting the Sinclair Refining Co. refinery at East Chicago, Ill., with Toledo, Ohio. Although these breaks did not occur in the watershed for any public water supply, the resultant death of fish and ducks on a scale large enough to provoke protest from numerous conservation groups, and the illness and even death of cattle drinking the contaminated water are convincing reminders of the menace in contamination of water by oil products.

Control

These dangers accompanying the pollution of water by petroleum products have led water works men either to resist the construction of pipelines across their watersheds altogether, or, where their legal rights to do so have not been clear, to insist upon adequate controls and safeguards. As a result of the negotiations between the Mahoning Valley Sanitary Dist. and the Sun Oil Line Co., in the above-mentioned case, a contract was drawn up, effective Feb. 21, 1931, for the construction and maintenance of the pipeline.

The Sun Oil Line Co. agreed:

1. To construct the 6-in. line of best quality, tested, seamless tube pipe with welded field joints, coated for protection against external corrosion
2. To encase the pipeline, at all road and stream crossings, within an outer 10-in. steel casing, properly sealed and vented above the surface of the ground in order to permit regular visual inspections
3. To maintain a regular patrol and inspection at least twice a week while the pipe is in operation
4. To move the line out of the district's watershed entirely if, in the opinion of the Ohio Dept. of Health, there occurred actual contamination of the water supply from a leak in the pipeline.

The Sanitary Dist. reports that no contamination has occurred to its water supply due to breaks in this line in the fifteen years since it was built, and that the company has adhered strictly to the provisions of the contract.

In the early part of 1946, the Sanitary Dist. was confronted with another request to construct a gasoline pipeline across its watershed, this time by the Sinclair Refining Co., which contemplated a line along a road that crossed the reservoir itself. After a review of pertinent experiences, including much of the material presented herein, the Board of Directors of the Sanitary Dist. refused to allow the line to cross district-owned property. When the company agreed to relocate the proposed line 2½ miles upstream from the district reservoir, however, the board, to be fair, decided to give the Sinclair Refining Co. equal treatment with the Sun Oil Line Co., and permit the new line to be built near the old one, on territory that was part of the watershed area, but not owned by the district. In return, a contract was drawn up in which the company recognized the rights of the district in the drainage area, and in which provisions similar to those in the earlier contract with the Sun Oil Line Co. were accepted.

Conclusions

That the existence of pipelines carrying petroleum products across water-

shed areas is a definite threat to water supplies seems certain. Although the number of such traversals is quite large, and the number of breaks comparatively small, the seriousness of the threat presented by even a minor break or leak is so great that the statistical risk is simply not worth taking if it can at all be avoided.

It would certainly seem that some sort of legislation to bring petroleum pipelines under more adequate control is needed. Perhaps this power should be given to the health departments of the various states, because of the danger to health that is involved.

In the absence of such legislation, however, and where outright refusal seems impossible or unwise, the operators of the pipeline should most certainly be bound, by contract, to recognize the rights of the water utility. Because of the difficulties in disposing of petroleum products, either by burning or by impounding and pumping into receptacles to be carted away, the line should be located as far upstream from the impounding reservoir or intake as possible. Furthermore, the operators should agree to construct the line of the best material available, to take all possible precautions in installation and to patrol and maintain the line effectively.

Acknowledgment

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Water Supply Practice in Germany

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As a member of a civilian team of technical investigators organized to serve under the Combined Intelligence Sub-committee G-2 of the Army and to report on new developments in German utility practice, the author had an unusual opportunity to observe water utility systems in that country immediately in the wake of the war. Some of these observations may interest American water works operators.

The investigations in Germany occupied 14 weeks starting on June 1, 1945. The targets visited included 14 public water systems and 4 large war production plants. Of the latter, two were munitions, one a chemical and one a synthetic rubber plant. Conferences were also held with consulting engineers, chemists, government officials, plant operators and equipment manufacturers.

War Maintenance and Operation

Considering the conditions under which German water utility systems were compelled to operate during the war, the general impression received was that the technical design and operation of these systems were good. As might be expected, the newer developments and the best equipment were observed in plants serving German war production. Apparently these plants were given relatively high priorities with which to obtain material and manpower. As in this country during the war, public utility construction was definitely limited to favor war production. There was evidence, however, that operators of these systems were supported by allotments of material and equipment for maintenance and emergency facilities, including those intended to camouflage and protect major units against bombing.

Vulnerability of Utilities

German water and power utilities are especially vulnerable to destruction by bombing. Often water from the various sources is brought to a central well adjacent to a low-lift pumping station. The housing over these structures on the "jugular vein" of the system is readily detectable from the air, and their destruction would have resulted in the immediate interruption of supply. Strangely, in spite of their vulnerability, selective bombing of central collecting well and pumping units in German water plants was not carried out by either the American or British Air Forces.

What destruction occurred in German water systems was especially heavy in the transmission and distribution systems. Much of this occurred

in the late months of the war when aerial bombing was most intensive. German water works operators expressed surprise that Allied air forces followed a policy of intensive bombing of war production plants when periodic destruction of water works intakes or pumping stations would have effectively stopped production at these plants. Such major war production items as synthetic gasoline, rubber, chemicals and munitions require heat exchange units which use large volumes of water. An interruption of supply would necessitate a shut down of operations.

Attempts to Camouflage Systems

Realizing the vulnerability of water supply and pumping units at such plants, the Germans resorted to various methods of deception. One, which was quite successful at two munitions plants, was to house the water pumping and treatment facilities in separate buildings which resembled typical German farm houses. Such plants were frequently built in wooded areas. If the utility buildings were exposed, they were so grouped and served by access roads as to resemble rural houses with their typical stone or stucco exteriors and red tiled roofs. In one acid recovery plant in a heavily wooded area, a steel stack was painted green and was equipped with a telescope device which would permit lowering to tree height when bombers approached. Buildings were half underground and half aboveground, with flat slab roofs covered with topsoil in which grass, shrubbery and small trees were planted for camouflage. Saw-tooth roof edges cast shadows similar to those of the nearby fir trees. These plants were operated partly by slave labor; and in the days following liberation, the freed laborers destroyed much operating equipment and many records. At the Bremen water works, camouflage of the open settling basins and slow sand filters apparently attracted the attention of our air reconnaissance, because this plant was heavily damaged by aerial bombing. At the time of inspection—July 1945—1 of the 2 settling basins, 11 of the 22 slow sand filters and 1 of the 2 filtered water storage reservoirs were out of service for repairs. Had it not been that about half of the water supply of Bremen is obtained through a large gravity transmission main from an impounded supply in the Harz Mountains, this city would have been almost entirely out of water for a long period. As discussed later in this paper, water service in that city suffered heavily both from Allied bombing and as a result of demolition by German army engineers.

Sources of Supply

In comparing German water works operation with American practice, one must consider that the per capita use of water in that country is low. It varies in the larger cities from 40 to 60 gpd., or about one-third the normal use in our cities. About 75 per cent of German public water supplies are from ground sources. These include deep and shallow wells, springs and infiltration galleries. Surface water is stored in open reservoirs typical of those used to supply water to large cities in New England. River supplies in Germany are filtered either naturally, by percolation through sand and gravel substrata to infiltration galleries, or artificially, in slow or rapid sand filters, usually without chemical treatment.

Wells

Much of Germany is underlain with an almost inexhaustible supply of cool,

clear ground water. The drain of war production on underground well supplies in that country was not as serious as in many local areas in the United States. Shallow wells predominate; usually they are hand or machine dug 40 to 60 ft. in depth. They are lined with stone, brick and pre-cast concrete. Before the war, well screens of copper alloy were most common. Since the war, because of the shortage of copper, screens have been made of steel covered with rubber, of fluted glazed ceramic material and of oak wood. Spacing of wells varies widely, depending on local conditions; the more frequent spacing ranges from 60 to 120 ft. At the head of each well is a valve box of brick or concrete with its locked cover flush with or just above the surface of the ground. Wells are connected by a cast-iron header system leading to a central collecting wall, from which the water is pumped for treatment. The header system is usually under negative pressure maintained by a vacuum pump, the head depending on the rate of withdrawal desired. Deep wells vary in depth from 300 to 1,500 ft. Usually they are operated as separate units. The deep-well pumps observed were of the series impeller and submerged pump and motor types.

In Germany, ground waters often contain objectionable amounts of carbon dioxide, iron and manganese in solution; and, therefore, must be treated. Carbon dioxide is removed by passing part or all of the water through tanks containing milk of lime. The Bucher process, in which hydrated lime is added to the reaction tank in proportion to the flow of the water to be treated, was used in most of the plants visited. The precipitated calcium carbonate is discharged intermittently from the settling cone to the sewer. Treatment to reduce carbon dioxide has been carried out in Germany since it was shown that the Leipzig lead poisoning outbreak was the result of action of water high in carbon dioxide on lead in service pipes. In this outbreak there were 50 severe and 200 mild cases of lead poisoning.

Soluble iron (2.0–0.5 ppm.) is reduced to 0.05 ppm. by aeration, settling and filtration. The water is pumped to various types of distributors from which it trickles down over staggered brick aerators about 7 ft. high and is then collected in settling basins either under or adjacent to the aerators. At intervals of about six years, the bricks are cleaned and the oxidized iron is removed by air drying and wire brushing. With from one to two hours' retention after aeration, 75 per cent of the iron is removed before filtration. Filters are of the rapid and slow gravity and pressure types, using sand as the filter medium. Rates of filtration are similar to those in American practice.

Manganese (1.0 ppm. and higher) is removed from German waters by filtration in gravity or pressure filters in which the filter medium is a calcined dolomite. It resembles crushed pottery clay, is very porous and consists of calcium and magnesium oxides and carbonates. The medium must be replaced about every six to eight years.

Infiltration Galleries

In Germany infiltration galleries are commonly used for the collection of water for public systems. They are of three types: (1) galleries adjacent to rivers or artificial recharge areas, (2) galleries cut into the sides of hills or mountains to intercept the flow of water through seams in rock, and (3) galleries in artificially built collecting areas between hills.

At Essen in the lower Ruhr Valley, the infiltration galleries are parallel to the river and about 30 ft. deep. The pipes are steel and reinforced concrete and are perforated or slotted to collect the water which infiltrates into the sand and gravel in which they are set. Part of the water reaches the galleries by natural infiltration from the river. To supplement this, long narrow basins of reinforced concrete have been built in the low ground back from the river which has settled because of coal removal from below. These open bottom basins are filled with from 4 to 6 ft. of sand and are flooded by direct flow from the river. In effect, they are slow sand filters built over or adjacent to the galleries. When the filters are clogged they are raked, and the dirty sand piled and removed. Replacement of filter sand is necessary at about eight-year intervals. Because of proximity to the Ruhr River, the area in which the filters and galleries at Stelle, from which water is supplied to the city of Essen and vicinity, had been heavily mined by the German engineers. Several water works employees, while working in the filter area, were killed by explosions of those mines.

At Hagen in the Upper Ruhr, a modern rapid sand filter plant was constructed to permit increased output from the infiltration galleries. By applying filtered water to the slow sand filters, it was possible to recharge the galleries at high rates and with less manpower. Prior to the construction of the rapid sand filter, difficulty had been experienced in getting enough water to the galleries by natural percolation from the river or by the slow sand filters during periods when algae were prolific in the river water. The temperature of the water in the infiltration galleries along the Ruhr fluctuates seasonally, being slightly less than that of the river.

The system of infiltration galleries which supplied water for the city of Munich is most interesting. About twenty miles south of the city, in the foothills of the Bavarian Alps, the city has developed a series of infiltration galleries in the Mangfall Valley. The collecting galleries are semi-elliptical in section and are about 3 ft. wide by 5 ft. high. They are constructed of open-jointed stone, brick, and also of concrete with slots in the sides and top, through which the ground water can enter the galleries. Some of the galleries are in the river valley and others are cut into the rock of the hills to intercept the underground flow. The water is clear and has a temperature the year round between 40 and 50° F. The water from one system—a series of radial galleries—is discharged into a central collecting well lined with white tile. Flow to the terminal storage reservoirs near the city is by gravity through steel and reinforced concrete transmission mains; it is measured and controlled by a Venturi chamber near the head works.

At the Ranna Works, supplying about half the water to Nuremberg, a novel infiltration gallery was recently built. A swamp caused by outcrop water between two hills about 30 miles northeast of the city was excavated to a depth of 25 ft., back-filled with gravel and covered by a concrete slab. In the gravel two 30-in. collecting pipes were placed to deliver the intercepted water to a storage reservoir and control works. Over the concrete slab sand and black soil were placed, and plantings made to conceal the collecting gallery. A supplemental supply was obtained from an infiltration tunnel

similar to those of Munich, extending 1,500 ft. into a mountain to intercept an underground flow of water discovered by exploration.

The quality of the water from the Munich and Nuremberg infiltration systems is good, and no further treatment except chlorination in the terminal reservoirs is necessary. The advantage of having a source of supply producing a clear and uniformly cool water, free of algae, is obvious. Unfortunately, time did not permit a study of the economics of the Munich and Nuremberg systems of infiltration galleries on which cost comparisons with other systems of surface storage could be made. The higher per capita use of water in the United States might make the cost of developing infiltration galleries too expensive for acceptance here. Nevertheless, this aspect of German water supply engineering is worthy of further study.

Surface Supplies

Storage of surface water collected from sparsely settled and wooded areas, as is practiced by New York and Boston, is also carried out in Germany; but before such water is supplied to consumers it is filtered. Examples of surface storage are the reservoirs of the Harz Mountain Water District, organized by the Province of Hanover to supply water to a score or more cities and villages between Österode and Bremen, and on the east side of the Province to Brunswick and many other villages along the route of the transmission main. The two dams are known as the Sesetalsperre and the Eckertalsperre. A private water company known as Wasserwerke Landkreiss Aachen has two impounding reservoirs in the hilly wooded area southeast of Aachen. From these reservoirs, water is supplied mostly by gravity to the coal mines and industrial area of 750 square miles east and north of that city. Through an elaborate system of pipes and storage reservoirs, water is supplied to more than forty villages and towns including such cities as Stolberg, Eschweiler, Elendorf, part of Aachen and even across the border into Holland. The Haspetalsperre Dam and Reservoir on the Ruhr, above the city of Hagen, is another example of storage of surface water for public water supply, as well as for control of the flow of that river for sanitation and power.

The concrete and stone dams built across the valleys to permit development of these reservoirs were well maintained. Because of the serious destruction to the Möhne dam on the upper Ruhr by the British air raid of May 17, 1943, the Germans had lowered the water level in all large storage reservoirs. At the time of the investigation, there was some concern that a deficiency in rainfall would cause a serious shortage of supply. The Möhne dam had been repaired, but still clearly evident was the destruction in the valley below, caused by the onrush of water released through the breach in the dam. At the Hagen filtration plant, below the Hengstey See and about 30 miles downstream from the Möhne Dam, the flood-water mark could be seen on the sides of buildings about 5 ft. above ground level.

Water Treatment

Rapid Sand Filters

German rapid sand filter plants of the gravity type were visited at Hagen, Wulzdorf near Bremenhaven, Österode, Stuttgart, and large war production plants at Schkopau, Leuna and Helsa.

With the exception of the one at Österode, these plants had been built within the past decade. They were well designed and equipped with modern control facilities. The most interesting features were the absence of overhead wash water troughs, the design of the filter bottoms and strainer systems and the use of air and water for backwashing.

In Germany, manufacturers design and build water works structures and facilities. The two most important types of rapid sand filters are known as the WABAG and the BAMAG types. Filter plants are constructed both with central and side galleries. The filters are from two to three times as long as they are wide, the narrow sides being parallel to the galleries. Raw water enters a long inlet channel on one side of each filter. When washing, this channel also serves as an outlet channel discharging to a sewer in the gallery or at the far side of the filter from the gallery. Between pairs of filters, there is also a common deep wash water channel. Since filters are about 8 ft. wide, the travel of wash water from the center of a filter to the wash water channel is not over 4 ft.

The WABAG filter bottom is made up of a series of pre-cast, reinforced concrete slabs about 6 in. thick, 18 in. wide and 36 in. long. Into threaded holes in the slab, the centers of which are spaced about $3\frac{3}{4}$ in. apart, special copper or glazed porcelain strainer heads are screwed. Each strainer has an extension nipple protruding into the space below the slab, a distance of about 6 in. When a filter is to be washed, compressed air is pumped into the space under the slab and enters the filter through the strainer extension pipe and head. Water is then pumped into the space below the slab and the filter is backwashed with a mixture of air and water. The air which is compressed in the upper part of the space under the slab enters the strainer extension pipe through a slot or hole in the side. Finally, to free the filter of entrained air, it is washed with water only. This type of filter has no gravel bed, the sand being placed directly over the strainers. The depth of sand is about 6 ft. The sand size varies from 1.0 to 0.6 mm. Such filters were being used with and without pre-treatment. At the Buna-Werke at Schkopau, where Salle River water was being treated, the settling time was only one hour, and no chemicals were used for coagulation. At other plants, chemicals were used only when the water was turbid. Often there was no provision for pre-settling. Operators reported freedom from mud ball difficulties and were well satisfied with results obtained from these filters.

The BAMAG type filters are similar to the WABAG plants in general layout and arrangement of inlet and wash water channels, but use a pipe manifold with porcelain nozzles for wash water. The distributor pipes are placed on coarse gravel about 1 ft. from the filter box floor. The pipe grid for the air wash is set in finer gravel about 10 in. above the wash water distributor grid; the two are staggered. The sand depth above the gravel is about 4 ft. Its size is similar to that in the WABAG filters. Design rates for both filters are similar to those used in American practice.

Rapid sand filters of the steel tub type, with mechanical rakes, were seen at Charlottenburg, a suburb of Berlin.

Slow Sand Filters

Slow sand filters were seen at Halle, the Probstheide and Thekla plants at

Leipzig, the Tegel and Müggelsee plants at Berlin, and at Bremen. Except at Bremen, the filters were covered and were operated to remove iron from ground water after aeration and settling. The filters at Bremen were open and treated Weser River water after 24 hours of settling, normally without chemical treatment.

The number of new steel pressure filters seen in Germany was especially surprising because of the reported shortage of steel during the war. They were used for iron and manganese removal as well as for roughing filters in lieu of settling, when production from rapid sand filters was restricted by algae in raw water supplies. Pressure filters were common in war production plants. They were single, double- and triple-decked, and ranged in diameter from about 3 to 6 ft.

Chemical Handling and Mixing

With the exception of that collected from infiltration galleries, most water supplied in the German cities visited was filtered. In general, operation and maintenance of these plants were good. The use of coagulants was exceptional, but several plants which normally do not coagulate the water prior to filtration use aluminum sulfate when the raw water is highly turbid. Only one plant used any other coagulant. This was at a munitions plant at Ebenhausen, where the Parr River water used for industrial purposes was coagulated with ferric chloride. At the same plant, well water for domestic use was treated with aluminum sulfate.

Chemical handling, storage, mixing, measuring and conveying practices in Germany are similar to those in the United States. Handling in bulk is done more by manual labor than by mechanical equipment. Cranes and conveyors were seen, but no vacuum systems. Batch mixing is common; mechanical stirring, general. Measurement is both gravimetric and volumetric, and rotameters are used in many plants. Duplicate equipment is not unusual. Rates of application in proportion to flow are controlled both automatically and manually.

Solution tanks are of steel, concrete and wood, with linings of hard and soft rubber, porcelain and wood. Chemical piping is made of lead, bakelite, plastics, hard rubber and steel with rubber or porcelain linings. Chemical feed pumps are lined with bakelite and plastic. At the Österode plant in the Harz Mountains, the chemical solution was filtered before being pumped to control devices.

Slow mixing (10–20 minutes) by vertical mechanical stirrers is used in the BAMAG type rapid sand filter plant at the Hagen Hengstey See plant. Mixing was accomplished at the Hagen Haspetalsperre plant of the WABAG type by the hydraulic jump principle. Around-the-end mixers are used at the Österode plant.

Softening

The waters in the German plants visited varied in hardness from 10–20 ppm. in the surface water to 500 ppm. in well and river water at Stuttgart. The range of hardness for most of the shallow well waters was between 100 and 200 ppm. No treatment plants for softening were seen except those for boiler feed water and special processing at industrial plants, where zeolite softeners were used.

Chlorination

German water works operators do not give the attention to chlorination that is considered good practice in the

United States. Ground waters are of good quality and do not require much chlorine. Usually from 0.1 to 0.2 ppm. of chlorine is applied to the water entering filtered water reservoirs. Residual chlorine is rarely found in distribution systems. Except for Stuttgart, where super-chlorination and de-chlorination is practiced, the largest amount of chlorine applied to filtered water was 0.6 ppm. at the Tegal plant in Berlin. It is customary at this plant to use ammonium sulfate with the chlorine to form chloramines. The ratio of chlorine to ammonia used was 2:1. An effort was made to carry about 0.4 ppm. residual chloramine in the reservoir from which water is pumped to the system.

Chlorination equipment is not as well housed or maintained as in the United States. Separate rooms for the chlorine and the chlorine control equipment were rare. Use of chlorine in large-capacity containers, equivalent to the ton containers so common in the United States, was observed at only two plants. Periodic weighing of cylinders, to check the operation of chlorine control equipment, was reported at only one or two plants. Hourly records of chlorine applied and residual tests are not customary. Duplicate chlorinating units were observed at several plants. Automatically controlled chlorinators were seen at one plant in Berlin and one at Österode. The use of rotameters to measure the flow of chlorine gas was observed at several plants.

The Stuttgart water works had the most interesting chlorination plant seen by members of the investigating party in Germany. Here super-chlorination (5.0 to 9.0 ppm.) is followed by de-chlorination through granular activated carbon filters. De-chlorination was being practiced at only one other plant—the drinking water system at the Buna-Werke at Schkopau—where the water was de-chlorinated by passing it through a granular carbon pressure filter.

Water Quality

The quality of German water supplies as delivered to the consumer is generally good, although in many cities water is quite hard. Free carbon dioxide in well waters often ranges from 40 to 60 ppm. It is removed or reduced by treatment with milk of lime, as previously described. Iron is common in well waters, ranging from 0.5 to 2.0 ppm. It is removed to less than 0.05 ppm. and often to 0.02 or 0.01 ppm. Manganese in well water may be as high as 1.0 ppm. and is almost entirely removed by treatment. The hardness of German water is largely temporary or carbonate hardness, although there are exceptions where sulfate hardness is high. Public water supplies vary in degrees of hardness from 10 to 500 ppm., the more common range being from 100 to 200 ppm. German water supplies investigated had a pH range of from 7.2 to 6.5.

Micro-organisms of the types experienced in this country are common in surface waters in Germany. Their growth is watched carefully and treatments are adjusted accordingly. At no city was the use of algicides reported. Copper and copper compounds are expensive in Germany and were not readily available during the past decade. Biologic pollution of German water supplies at the source is not heavy, except where river water is used. Even without chlorination, the quality of water produced at most plants after filtration is bacteriologically of good quality. In the plants of the larger

cities, chemical and bacteriological laboratories are maintained and the technical control over the treatment is good. Public health officials also collect and analyze samples from the plants and the distribution systems.

Transmission Mains

German transmission mains are constructed of steel, cast iron and reinforced concrete; and gravity lines are of pre-cast and cast-in-place concrete and three-ring brick. Sizes varied from about 12 to 72 in. in effective diameter. Most German cities have multiple sources of supply and therefore many transmission mains. This gives wide flexibility in operation, an especially desirable condition in wartime. In spite of this, however, destruction of transmission mains by aerial bombing was heavy. At Leipzig, the Carnitz and Thallwitz plants in the Russian zone of occupation and the Naunhof plants in the American area could not supply the city because of damage to transmission lines by bombs dropped by American airmen just prior to occupation late in April. Fortunately, the reservoirs at the Probstheide plant in the city were full prior to the bombing. By rationing of water and use of supplemental supplies pumped in from suburban systems and emergency wells constructed at numerous points in the city, complete failure of water service was avoided. In addition to damage to the water transmission mains at Leipzig, bombs damaged the main pressure sewer to the broad irrigation sewage farms in the suburbs; and for many weeks it was necessary to discharge raw sewage to the Elster River.

In Bremen, destruction of transmission mains was especially serious. The water filtration plant is in Neustadt, across the Weser River. Five cast-iron and steel transmission mains crossed the river on three bridges. In addition, one large steel transmission main was laid under the river. As a result of American bombing, two of the bridge lines and the one under the river were broken and rendered useless. In demolishing the bridges, the German engineers destroyed the other three mains, leaving the city without water until American military engineers suspended steel mains across the demolished bridge spans.

So intensive was the bombing in Nuremberg that all but one of six transmission mains were out of service when the American Army entered the city about the middle of May.

Storage Reservoirs

Water storage reservoirs in Germany are usually covered and well concealed. They are built of brick and concrete in multiple units. The newer ones at Munich, Nuremberg and Kassel are most interesting, and in appearance surpass any the author has seen in the United States. The 262-mil.gal. Kreupullach Reservoir at Munich, built in 1937, is especially attractive. In the white tiled control room there is an electrically illuminated wall diagram which shows by colored lights what valves are open, the elevation of water in each unit and the flow in and out of the reservoir. The interior finish of the reservoir is white. Ventilation is good and condensation at a minimum. An inspection gallery lined in white tile crosses the reservoir. Lights with white reflectors are numerous; when they are turned on, the deep blue color of the water has a striking effect. Valves, gates, doors, and operation mechanisms are of polished non-rusting steel. What a contrast they present to the reservoirs—dark, damp, inaccessi-

ble, rust-streaked and poorly ventilated —so common in this country!

Drainage from this reservoir was not wasted to the nearby stream, but was discharged to a concrete basin in the valley, from which the water was discharged to wells to recharge the underground supply. Floating islands of logs covered with top soil and small bushes served to camouflage the open receiving basin by minimizing the surface area of the water. The underground Venturi and valve control chambers were also well camouflaged. They had ground-level access doors which were counterweighted to open automatically when unlocked. Visitors were not permitted to enter the Schumhausenbuch Reservoir at Nuremberg unless they wore felt shoes over their regular ones. A bomb dropped on the roof of this reservoir sheared a hole 50 ft. in diameter through the roof.

Elevated Tanks

Elevated steel tanks enclosed in stone and brick towers are common in Germany. Many are artistically designed. Booster pumping stations at the street floor, with quarters for housing employees or for supply storage on upper floors, are not uncommon. Many were seriously damaged by bombing.

Pumping Equipment

In German water works, steam- and electric-driven pumping units of many types are used. In the low-lift pumping plants visited, more old steam crank and flywheel and triple-expansion pumping engines were used than is customary in the United States. High-pressure pumping equipment is of much more modern design. Motor- and steam-turbine-driven centrifugal pumps are common. The newer stations are well laid out and are equipped with modern control and recording equipment. In Berlin, standby diesel engine generators are used at one of the large electric stations. Both hard coal and brown coal briquettes were used for fuel. Much more hand-firing of boilers was observed in the steam plants than in America. The larger stations are equipped with stokers. Stacks at power plants were not damaged by bombing unless there was a direct hit.

Heavily reinforced concrete shells in inverted "U" and "V" shapes were set over pumps and motors in order to protect them against the falling debris of bombing. Single and double heavily reinforced concrete housings, cylindrical and rectangular in shape and equipped with steel doors, were built in the operating rooms of water treatment and pumping plants to protect employees on duty. Special outside or underground air raid shelters or "bunkers" were built for the protection of other plant employees during periods of bombing.

Distribution System

The distribution systems of German water works are about 90 per cent cast iron. During the war, steel was used to a considerable extent for repairs. The sizes vary from about $3\frac{1}{8}$ to 39 in. in diameter, with the 4 to 10 in. range most common. Pressures in the system vary from 25 to 70 psi. Damage to the distribution systems was extensive, and temporary service through fire hose and rapid joint steel pipe laid on the surface was resorted to after air raids.

At Essen, where aerial bombing was heavy, 2 per cent of the entire water distribution system was destroyed by bombing. Many sections of bombed cities were out of water for weeks. In

TABLE 1

War Damage to German Water Distribution Systems

City	Pipe in System *km.*	Breaks in System
Essen	1,000	2,000
Munich	1,700	1,860
Bremen	1,200	1,200
Nuremberg	670	1,200
Halle	362	200
Hagen	60	200

general, the extent of damage to water mains and sewers in German cities, with so little residual chlorine being carried in the water in the distribution systems, makes it seem surprising that water-borne epidemics did not occur in most large cities. Water was transported in tank wagons and consumers were required to carry water in pails from temporary supply stations.

Cast-iron pipe fractured easily in the area affected by bombs, but breaks usually stopped at joints. German utilities carried large stocks of cast-iron and steel pipe for emergency repairs. A special mechanical joint with a rubber gasket permitted quick replacement of damaged cast-iron pipe. This pipe is very popular with German water works engineers and has much to commend it to American engineers. Steel pipe was crumpled, twisted and ruptured by the impact of bombing. At Munich it was reported that, even after repairs had been made to steel pipe, leaks were found at various distances back from the bomb crater which required subsequent excavation for pipe repair or replacement. Apparently when a direct bomb hit was made, a shock wave transmitted by the water created stresses which ruptured the pipe some distance from the point of impact.

Some idea of the damage which bombing caused to German water distribution systems may be obtained from Table 1.

Hydrants and Valves

The most common fire hydrant used in Germany is the underground type. Access to the outlet and control valve is made possible by removing a cast-iron cover set in a frame flush with the pavement. An extension pipe $2\frac{1}{2}$-in. to 4 in. in diameter with a bayonet-type fitting is slipped over the outlet and, by twisting, is held firmly in place. The valve to allow flow to the hydrant is opened by a long key. Fire hose connections are made to the extension pipe which protrudes about 3 ft. above ground. When service is no longer needed, the valve is closed and the extension pipe removed. In cold climates, these hydrants are difficult to maintain because the cover tends to freeze in place.

German fire department officials prefer the aboveground hydrants, which are similar to American makes. It was observed that the aboveground hydrants are numerous at all large German war plants. They were used around such new buildings as the I. G. Farben office building in Frankfurt which was occupied by SHAEF and later by ETOUSA. These hydrants were also seen in the high value property areas of large cities. German water works engineers expressed the opinion that these aboveground hydrants would eventually replace the underground hydrants in large cities, even though their cost is two to three times that of the older hydrant.

Valves in German water systems are well made and similar to those of American manufacture.

The location of both valves and hydrants is well identified in Germany by signs posted on the walls of nearby

buildings. Enameled signs similar to automobile plates show by letters, arrows and numbers the exact location of underground operating facilities and the size main they serve. At plants where multiple water systems are used, signs of various colors and borders serve to identify the different systems. Manhole covers are also identified by special markings in the castings.

Service Pipes and Meters

Service pipes in Germany are of lead and galvanized iron. Copper is too rare for such use. All services are metered. Discs and sleeve type meters are common on domestic services. Inline rotary meters are used on larger ones. Plastic and pressed fiber discs, rotors and gears are much used.

Summary

A general summary of German water works practice in 1945 records that:

1. Considering war conditions, German water utilities were well maintained and operated.

2. German public and industrial water utilities were vulnerable, but in general, they were not made special targets in American and British bombing attacks.

3. Destruction of German transmission and distribution piping by aerial bombing was very heavy, especially in the later months of the war.

4. German water works designers and operators gave special attention to camouflaging plants and to protecting vital equipment and operating personnel against aerial bombing.

5. About 75 per cent of German public works systems depends on ground water for its source of supply.

6. Treatment for removal of carbon dioxide, iron and manganese is common in Germany.

7. The Germans use various types of infiltration galleries to obtain water for public supplies. The design and economics of these systems are worthy of study.

8. The design of German rapid sand filter plants, especially the use of air and water for filter washing, the design of inlet and wash water channels, and filter underdrains systems, are most interesting.

9. Chlorination practice in Germany is not up to American standards.

10. Some of the newer storage reservoirs seen in Germany were superior in design and equipment to equivalent American structures.

11. In the large cities in Germany, pumping equipment compares favorably with American standards. Much old steam pumping equipment was being used.

12. Distribution mains were seriously damaged by bombing. A special mechanical cast-iron pipe joint was very effective in making emergency repairs.

13. While underground fire hydrants prevail in Germany, the trend is toward aboveground hydrants.

Acknowledgments

While the author served as a member of the War Utilities Sub-committee, Technical Industrial Intelligence Com., CIOS. G-2, U. S. Army, in making many of these surveys, he was accompanied and assisted by Dr. Anthony J. Fischer and Dr. Arthur V. Sheridan of the Safety and Technical Subcommittee. Later the TIIC. team was supplemented by Lt.Col. Joseph J. Gilbert, San. Corps, who represented CIOS. and the Surgeon General's Office, U.S. Army, and was assisted by Major Myron W. Tatlock and Lt. Harold P. Pfreimer, both affiliated with the Allied Military Government in Germany.

The Mode of Action of Chlorine

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A contribution to the Journal

EDITOR'S NOTE: *This paper is a most important addition to the literature of water supply. Employing the technics of physiological chemistry to determine the manner in which chlorine exercises its bactericidal action, the authors show that the trace level at which chlorine is effective implies that it must inhibit a key enzymatic process. This process is determined to be the oxidation of glucose by the bacterial cell; once the power of glucose oxidation is lost, the bacterial cells die—the suspension becomes sterile. The authors further show that the reaction is not reversible; that is, that bacteria once inactivated by chlorine cannot be reactivated. They show that another halogen—iodine—has the same power, but requires higher concentrations for equivalent bacterial kills.*

Their second paper, which follows (p. 1306), entitled "A Note on an Enzymatic Method of Estimating Chlorine," records a method of determining available chlorine which is based upon this inhibition of enzymatic action.

It is hoped that the authors may choose to extend their studies to determine what effects if any are produced in the human digestive system by drinking water which is treated with the conventional doses of chlorine.

AT the request of the Office of the Quartermaster General, the authors undertook from May 1944 to December 1945 a study of the "fundamental biochemistry of water disinfection" under contract OEM-cmr 443. A full account of the experimental work is given in the bimonthly reports submitted to the Committee on Medical Research and in a communication to the *Journal of Bacteriology* (1). It is proposed to provide here a detailed summary of the experimental work and some of the conclusions to be drawn therefrom.

Enzyme-Trace Substance Theory

The most remarkable feature of the bactericidal action of chlorine is the low concentration in which it is effective. From 0.2 to 2.0 ppm. chlorine are adequate to sterilize completely water which is not grossly contaminated with organic, nitrogenous material. Biochemical action at what may be called the trace level of concentration is most readily explained in terms of effects on enzyme systems.

There is a theory, known as the enzyme-trace substance theory (2), which postulates that any substance, natural or synthetic, which in trace amounts has a profound influence on biological processes, must either be an essential part of some enzyme system or inhibit or modify some enzyme reaction. There are many instances of the validity of this theory. Thus four

of the vitamins—riboflavin, nicotinic acid, pyridoxine and thiamine—have been shown to be essential parts of enzyme systems. Several of the trace metals, such as copper, zinc, iron and manganese, have been identified as prosthetic groups of essential enzyme systems. Two of the hormones—insulin and a factor from the pituitary gland—have been implicated as regulators of the action of hexokinase, an enzyme which controls the conversion of sugar to glycogen. Among the drugs and chemotherapeutic agents, phloridzin, gramicidin, sulfonamides, cyanide, fluoride, physostigmine, fluoroacetate, strychnine and eserine have turned out to be highly specific inhibitors of enzyme reactions.

Inhibition of Glucose Oxidation

The trace level at which chlorine exerts its bactericidal action suggested an explanation in terms of inhibition of some key enzymatic process. Exploratory experiments soon disclosed that there was a precise parallelism between the effect of chlorine on bacterial growth and its effect on the rate at which glucose is oxidized by the bacterial cell. A concentration of chlorine which was just enough to stop the oxidation of glucose completely was also just adequate to suppress growth; whereas growth, albeit delayed, always took place when inhibition of glucose oxidation was less than 100 per cent.

This parallelism was confirmed with various organic chlorine compounds, such as halazone and succinchlorimide, with various bacteria in addition to *Esch. coli* and with different concentrations of bacteria in given tests. All that had to be determined with accuracy was the minimal amount of chlorine necessary to prevent a given amount of bacteria from oxidizing glucose, and that concentration invariably was found to sterilize the suspension. After carrying out some five hundred experiments of this nature without finding a single exception, there could be no question of chance coincidence. The evidence is clear that suppression of glucose oxidation is causally connected with cessation of growth.

Confirmatory evidence also came from experiments in which the time of exposure of bacteria to chlorine was varied. A series of identical bacterial suspensions was treated with a given amount of chlorine for varying lengths of time (0.5 to 20 minutes) and the treated suspensions, after neutralization of the chlorine, were then tested for their ability to oxidize glucose and for their viability. After 0.5 minute's exposure, the bacterial suspension still oxidized glucose but at only 5 per cent of the original rate. Consistent with that oxidizing ability, the suspension was still viable. After 5 or more minutes' exposure to the same concentration of chlorine, however, all power to oxidize glucose was lost and simultaneously the suspension became sterile.

Localizing Chlorine Action

Triosephosphoric Dehydrogenase Catalyst

The oxidation of glucose is a complicated process involving about twenty different enzymes. The task involved was to localize the action of chlorine by determining which one or ones were primarily affected by chlorine. On the basis of the known properties of the constituent enzymes, the so-called triosephosphoric dehydrogenase was considered to be the most likely "Achilles heel" of the entire process. This enzyme catalyzes an oxidation reaction which is essential to the entire process; that is to say, the oxidation of glucose will stop as soon as the triosephosphoric dehydrogenase is prevented from

functioning. This enzyme is widely distributed in animal and plant tissues and in bacteria. It can be prepared readily from skeletal muscle and yeast. The isolated and purified rabbit skeletal muscle enzyme was found to be about five times more sensitive to chlorine than was the process of glucose oxidation in the intact bacterial cell.

An increase in sensitivity of the isolated enzyme, compared to its sensitivity in the intact cell, is precisely what would be expected if the triosephosphoric enzyme were the site of action of chlorine. Proteins in general react nonspecifically with chlorine and thereby reduce the amount available for reaction with enzymes. Obviously the ratio of total protein to enzyme protein will be much higher in the intact cell than in the purified enzyme preparation and, as this ratio becomes smaller with purification of the enzyme, an increasingly higher proportion of the added chlorine becomes available for reaction with the enzyme. In other words, although the sensitivity of the enzyme to chlorine does not change, a smaller proportion of added chlorine is available for reaction with the enzyme in the intact cell than in the isolated and purified preparation. The apparent increase in sensitivity to chlorine with purification becomes, in effect, a measure of the degree to which chlorine is dissipated in side reactions with proteins generally.

Oxidation of Triosephosphoric Acid

The experimental data are consistent with the position that chlorine is bactericidal by virtue of its ability to paralyze the oxidation of glucose at the point in the fermentation cycle where triosephosphoric acid is oxidized to phosphoglyceric acid. The question may well be raised whether it is justifiable to explain the bactericidal action of chlorine exclusively in these terms. At the trace level of concentration in which chlorine is bactericidal (0.2–2.0 ppm.) a considerable number of enzymes become inactivated. Thus, the triosephosphoric enzyme is not unique in this respect; in fact it shares this property with a whole class of enzymes which contain sulfhydryl groups essential to their activity. When these sulfhydryl groups become oxidized by chlorine, enzymatic activity is abolished irreversibly.

The decision to attribute the action of chlorine exclusively to the inhibition of the triosephosphoric enzyme is based not on the uniqueness or specificity of this particular inhibition, but rather on the essential nature of glucose oxidation and the key role of the triosephosphoric enzyme in the over-all process. All bacteria with the exception of certain spore cells are able to oxidize sugar, and wherever the relationship has been tested, their ability to grow has been found to be contingent upon their ability to oxidize or ferment sugar. In other words, the glucose oxidizing system is a universal constituent of the enzymatic equipment of bacterial cells. It thus becomes possible to restrict consideration of the action of chlorine to a sensitive spot in the glucose-oxidizing mechanism, even though other enzyme systems are known to be affected by chlorine. These other systems are not comparable with the glucose-oxidizing mechanism either in their essential nature or their distribution in bacteria generally.

The authors' observations on the effects of chlorine on spore cells are germane to the above discussion. From 10 to 20 times as much chlorine per milligram dry weight of cells is needed to sterilize spore cells as is needed for vegetative cells. If the viability of spore cells were contingent upon the

power of oxidizing glucose, they should be just as sensitive to chlorine as vegetative cells. Experiment has shown, however, that various viable spore cells do not possess the ability to oxidize glucose and, furthermore, that the viability of spore cells which can oxidize glucose is not dependent upon this power. Thus, from 0.2 to 2.0 ppm. chlorine completely inhibit glucose oxidation by spore cells without influencing their viability.

The fact that the oxidation of glucose in spore cells is inhibited by the same chlorine concentrations which are bactericidal proves conclusively that the peculiar insensitivity of spore cells to chlorine is not due to their impermeability to chlorine but, rather, to the fact that the systems essential to viability are not the same as those of vegetative cells and are not as sensitive. It would be beyond the scope of this discussion to consider which type of enzymatic system is essential for the viability of spore cells, but certainly the glucose-oxidizing system can be excluded.

The triosephosphoric dehydrogenase is known to be inhibited by reagents other than chlorine. If the basic assumptions as to the mode of action of chlorine are valid, it would follow that the same parallelism between inhibition of growth and inhibition of glucose oxidation should obtain for other inhibitory agents. This prediction has been checked with two types of inhibitors known to inhibit very effectively the triosephosphoric enzyme, *viz.*, iodoacetic acid and the organomercurials. With both these reagents, whenever the concentrations were adequate to inhibit completely the oxidation of glucose, viability was at the same time eliminated. Indeed, a perfect parallelism of effects was observed. There were, however, two respects in which the mechanism of action of chlorine differed from that of iodoacetic acid or organomercurials. First, the action of chlorine was almost instantaneous, whereas that of iodoacetic acid or the organomercurials was not complete before at least an hour. Second, the action of chlorine was irreversible. That is to say, the damage was done whether or not excess chlorine was neutralized. With the other two, however, the bactericidal action required the continued presence of the reagent throughout the test. In other words, organomercurials and iodoacetic acid are bacteriostatic agents whose effects are strictly reversible. The action of chlorine, by contrast, is almost instantaneous and, as far as is known, irreversible.

Both iodoacetic acid and organomercurials are capable of combining with the sulfhydryl groups of proteins, and presumably their inhibitory effect on the triosephosphoric enzyme is related to this property. By adding other sulfhydryl-containing substances like cysteine or glutathione, it is possible to reverse the inhibiting effect that either iodoacetic acid or the organomercurials have on the triosephosphoric enzyme. It is significant that, when the oxidation of glucose is allowed to proceed in this manner, the inhibition of growth is simultaneously eliminated.

Action of Other Halogens

Thus far, only one of the halogens—chlorine—has been considered, and it may well be asked whether the same considerations apply to bromine and iodine. The authors have not carried out any experiments on the bactericidal action of bromine, but some data is available on the mode of action of iodine. Precisely the same parallelism obtains between the effect of iodine on

glucose oxidation and the effect on bacterial growth. That is to say, both iodine and chlorine primarily affect the glucose oxidation system. Higher concentrations of iodine are needed, however, to inhibit the isolated triosephosphoric enzyme than are needed to inhibit glucose oxidation in the intact bacterial cells. This discrepancy argues that iodine is inactivating some key enzyme of the glucose oxidation system other than the triosephosphoric dehydrogenase.

Some years ago, during a systematic study of zymohexase (3), an enzyme which splits the phosphorylated sugar molecule into two 3-carbon units, it was observed that a few parts per million iodine could inactivate the enzyme. This particular enzyme is an essential component of all glucose oxidation systems regardless of origin, and it may well be the primary point of attack of iodine. The fact that iodine does not act in precisely the same way as chlorine may be of special significance from the standpoint of the amebic cyst. The work of Prof. Gordon Fair and his group at Harvard Univ. has established the superiority of iodine over chlorine as a cysticidal agent. Perhaps iodine is capable of paralyzing some enzyme system essential to viability in the cyst but relatively unaffected by chlorine.

Disinfection of Water

Some aspects of this investigation which are relevant to the problem of water disinfection merit consideration. A great deal of effort has been extended in the direction of preparing organic compounds containing active chlorine, presumably with the hope that a compound will be found which will be a more effective bactericidal agent than chlorine. In the light of the present investigation, it would appear that chlorine and organic compounds containing active chlorine act in precisely the same way, since hypochlorite is the active principle in each case.

These various compounds do not differ in mechanism of action but only in the speed with which they liberate active chlorine. A compound like halazone conserves active chlorine in the sense that it penetrates the bacterial cell before its active chlorine can become dissipated by interaction with the proteins of the bacterial cell. For that reason halazone appears to be more active in terms of active chlorine than chlorine itself. In other words, halazone and comparable compounds conserve active chlorine but are incapable of doing more than chlorine itself.

It is instructive to consider that the most effective antiseptics yet discovered—chlorine, iodine and the organomercurials—are all active by virtue of their effects on enzyme systems. In the enzyme laboratory of the Dept. of Medicine of the Columbia Univ. College of Physicians and Surgeons, Capt. W. E. Knox and his group have succeeded in demonstrating that the class of cationic detergents are also highly effective enzyme inhibitors, and work is in progress to elucidate the precise point of action of these reagents. Certainly the development of new antiseptic agents must await the discovery of new inhibitors of enzyme reactions vital to the growth of bacteria.

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A Note on an Enzymatic Method of Estimating Chlorine

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A contribution to the Journal

EDITOR'S NOTE: *This investigation was carried out under contract with the Com. of Medical Research of the Office of Scientific Research and Development and at the request of the Office of the Quartermaster General. Dr. Stumpf is now at the School of Public Health, Univ. of Michigan, Ann Arbor, Mich.*

CHLORINE and compounds containing active chlorine irreversibly inactivate enzymes whose activity depends upon the integrity of certain groups sensitive to oxidation. These groups are generally assumed to be sulfhydryl. The sterilizing action of chlorine-containing compounds has been shown (1) to be due primarily to the irreversible destruction of the triosephosphoric dehydrogenase which is essential for the oxidation of glucose and, hence, the growth of the microorganism.

The discovery of the mode of action of chlorine has an interesting application to the practical problem of determining the concentration of active chlorine in drinking water. The usual chemical methods (2) fail to differentiate between active chlorine (as Cl_2 or HOCl) and bound chlorine (as chloramines). Dr. Gordon Fair pointed out to the authors the possibility of using enzymes as reagents for measuring the concentration of active chlorine in drinking water. The sensitivity of the triosephosphoric enzyme to chlorine in very low concentration at once suggested the use of this enzyme for the purpose. The enzyme and the catalytic system in which it works are not readily prepared, however, and the method of testing would be beyond the capacity of the average water-sterilizing plant.

The authors therefore investigated the use of the proteolytic enzyme, papain, as a reagent for active chlorine. In addition to the fact that it is readily available commercially, the method of testing for its activity is extremely simple. Milk incubated with papain undergoes clotting, and the clotting time becomes a measure of the concentration of papain. In order to detect chlorine at the concentrations found in drinking water, the enzyme must be obtained in highly purified form so that the chlorine is not dissipated in reactions with protein impurities.

Experimental

Preparation of Enzymes

Papain (Merck) is ground up with 10 volumes of water, and the suspension is centrifuged to remove insoluble

particles. The turbid supernatant fluid is mixed with $\frac{1}{6}$ volume of 25 per cent basic lead acetate and the copious precipitate is discarded after centrifugation. The supernatant solution is dialyzed against running tap water for 12 hours. Alumina Cγ gel is added in an amount which removes enzymatically inactive protein without adsorbing papain. The exact quantity of alumina Cγ (3) to be added must be determined in a pilot run. The light straw-colored solution of papain which results is lyophilized and the dry powder can be stored indefinitely.

Activator of Papain

Sulfite has been found to be a more satisfactory activator of papain than thiosulfate. It has been exclusively used in the method described below.

Tests

When determining from 0.14 to 1.4 ppm. of chlorine, the procedure is as follows: 0.02 ml. of an 0.8 per cent papain solution are mixed with 0.05 ml. of M/5 phosphate buffer of pH 6, varying amounts of chlorine (as NaOCl) and distilled water to make a total of 1 ml. The mixture is allowed to stand 5 minutes at 25° C., after which time 0.1 ml. of a freshly prepared and neutralized solution of sodium sulfite (0.05 per cent) is added. The tubes in which the test is carried out are placed in a water bath maintained at 57° C. After 5 minutes' incubation, 2 ml. of 20 per cent milk, also at 57° C. (Dryco powder suspended in M/10 acetate buffer of pH 5.5), are rapidly added. The clotting time is then noted. Table 1 shows the relation between the concentration of chlorine and the degree of inhibition of papain action. From the data a standard curve can be constructed which will permit reading off the chlorine concentration, given the experimentally determined clotting time. It is to be noted that the chlorine concentration referred to in the tables is that of the unknown solution after mixing with the papain and buffer solutions, but prior to mixing with the sulfite and milk solutions.

TABLE 1

Effect of Chlorine Concentration on Inhibition of Papain Action

Concentration of Chlorine *ppm.*	$\frac{1}{\text{Clotting Time}}$ *minutes*	Inhibition *per cent*
0	0.72	0
0.14	0.60	16
0.35	0.36	50
0.7	0.25	65
1.0	0.14	80
1.4	0.12	83

When determining from 1 to 10 ppm. of chlorine, the procedure differs in that (a) 0.2 ml. of 0.8 per cent papain and (b) 0.2 ml. of the sulfite reagent are used instead of the quantities given above, and (c) the clotting reaction is carried out at 37° C. Table 2 provides the data for constructing the standard curve for this particular range of chlorine concentrations.

TABLE 2

Effect of Chlorine Concentrations of 1–10 ppm. on Inhibition of Papain Action

Concentration of Chlorine *ppm.*	$\frac{1}{\text{Clotting Time}}$ *minutes*	Inhibition *per cent*
0	1.5	0
0.7	1.1	27
1.4	0.86	43
4.2	0.43	71
8.4	0.24	85

Measurement of Clotting Time

The milk is prepared by suspending 20 g. Dryco powdered milk in M/10 acetate buffer of pH 5.5. The suspen-

sion is then centrifuged and the supernatant fluid is used for the experiment. When stored at 4° C., the prepared milk will keep indefinitely. The tests are carried out in tubes 14 mm. (internal diameter) and 150 mm. in length. All the solutions used must be brought to temperature equilibrium before the test is begun (at least 5 minutes' incubation in the water bath). To observe clotting time, add milk and shake the whole tube very gently in the bath. The end-point for clotting is the time when the thin film of milk which wets the side of the test tube curdles. This is an almost instantaneous reaction and the time interval is very reproducible. Clotting time should vary from not less than 30 seconds to not more than 6 or 7 minutes.

Conditions for Maximum Sensitivity

The standard curves set up from the data in tables 1 and 2 are curvilinear. In other words, the slope falls rather sharply over the range from 60 to 10 per cent inhibition. This means tha estimations of chlorine at the platea of the curve are relatively inaccurate It is therefore advisable to make meas urements between 0 and 60 per cen inhibition; within this range the curv is fairly linear.

The authors have preferred to de scribe a method in which the reagent other than the milk are used in quanti ties of less than 0.1 ml. By appropriat dilution of the enzyme, phosphate an sulfite solutions, larger volumes may b pipetted without affecting the accurac of the chlorine estimation.

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Ground Water Conditions at Charleston, W. Va.

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A contribution to the Journal

CHARLESTON, W.Va., is the center of a highly industrialized area which extends along the Kanawha River from Belle to Nitro. The population of Charleston is now more than 60,000, and the surrounding area has approximately an equal number of inhabitants.

As a part of the state-wide investigation of ground-water resources by the West Virginia Geological Survey in co-operation with the U.S. Geological Survey, several weeks during 1941, 1942 and 1943 were spent in field investigations of the ground-water conditions in and near Charleston. Supplementary investigations have been made irregularly since that time. About 40 wells in Charleston and a similar number in the surrounding area were visited, and data obtained on the occurrence, quantity and quality of the ground water. The investigations are being continued, and this report merely summarizes the information now available. The conclusions are tentative and will be checked by additional detailed study.

These brief reconnaissance studies indicate that the increased withdrawals of ground water during the past few years have seriously decreased the yield of some older wells. Little reliable information is available on the fluctuation of the water levels in wells, but the deepening of many wells and the abandoning of others suggests that the available supply of ground water of a satisfactory quality may not be adequate for further development in all parts of the city.

Geology and Ground Water

Terrace Alluvium

Charleston lies along the river terraces adjacent to the Kanawha River. These terraces are underlain by alluvial clay, silt, sand and gravel deposits that average about 50 ft. in thickness. Figure 1, based on the Kanawha River charts of the U.S. Engineers, shows a geologic section across the river near Belle, W.Va., and Fig. 2 shows a similar section along the foundation of the bridge over the Chesapeake & Ohio Railroad at the Carbide and Carbon Chemical Co. plant at South Charleston, as based on data of the West Virginia Highway Dept. The material is largely clay or sandy clay, but moderately coarse sand or gravel occurs generally at the base, as shown in Table 1. Locally, the lower few feet contain partly consolidated cherty gravel. Available records of materials encountered in the wells do not furnish de-

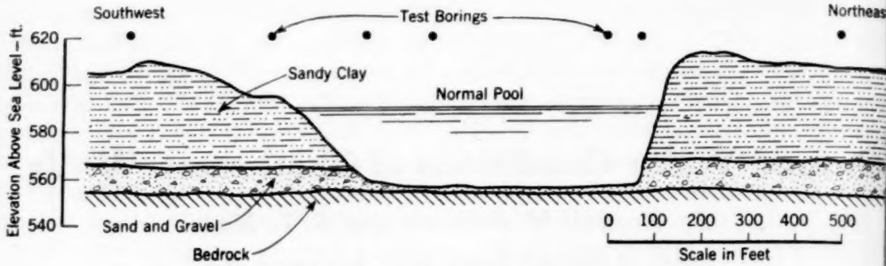

FIG. 1. Geologic Section Across Kanawha River Near Belle, W.Va.

TABLE 1

*Typical Section Through the Terrace Alluvium**

Material	Thickness *ft.*	Depth *ft.*
Clay and sandy clay	18.7	18.7
Sand	11.0	29.7
Sand, micaceous	10.0	39.7
Sandy clay	2.2	41.9
Sand, coarse micaceous	4.1	46.0
Sand and gravel, partly consolidated, containing fragments of chert and carbonized leaves and wood	5.7	51.7
Sandstone (bedrock)	—	—

* Generalized log of test borings at site of Chesapeake and Potomac Telephone Co. Building, Charleston. The average water level was 29.5 ft. below the land surface.

tailed information on the character of the alluvial deposits throughout the city, but it is evident that the character of the alluvial deposits varies considerably.

Most of the well logs in Charleston do not indicate that appreciable quantities of water were encountered in these sand deposits, but a few of the better wells do rely upon water in these terrace deposits. Other wells were drilled into bedrock during the war because screens were not available. About 2 mgd. were pumped from 7 test borings during the construction of the Chesapeake & Ohio Building (1), and relatively large quantities of water were encountered also during the construction of other large buildings, such as the Kanawha Valley Building. Moderately large supplies of water are obtained from the thick terrace alluvium on Blaine Island and at the Ohio-Ape plant in Nitro.

It is reported that the water level in the wells of the Charleston Laundry rose about 8 ft. following the increase in the pool stage of the Kanawha River (2). This suggests that it may be possible to locate wells so as to permit river water to enter the aquifer and replace a part of the water withdrawn. Wells depending partly upon recharge from the river are relatively common along the Ohio River (3).

These records indicate that the terrace alluvium will yield important supplies of water to properly constructed and developed wells in some localities in Charleston, but that these deposits are too fine to yield important quantities of water in some other localities. It appears that the supplies of water in the alluvial terraces have not been utilized fully because their water-bearing properties have not been realized adequately and the installation of screens has not been understood.

Consolidated Rock Formations

The consolidated bedrock strata beneath Charleston are, largely, alternating sandstones and shales, with minor beds of coal and limestone. The strata under most of the city compri

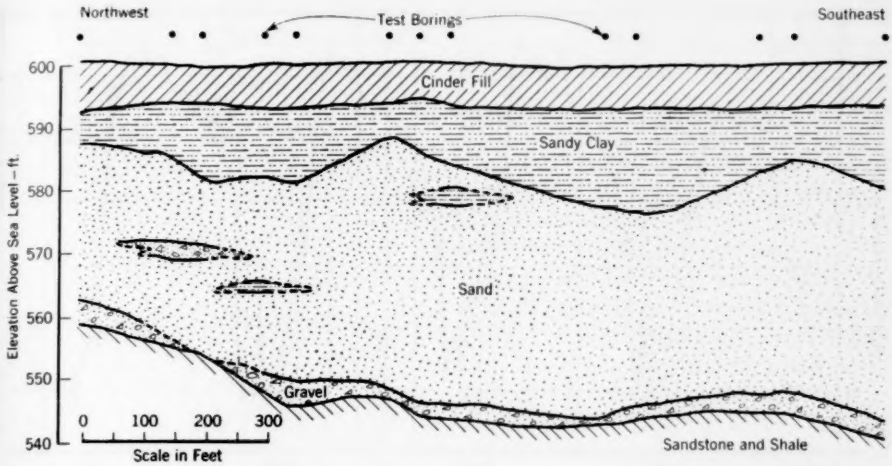

FIG. 2. Geologic Section Along Foundation of Bridge at South Charleston, W.Va.

TABLE 2

*Typical Section Through the Bedrock Strata**

Material	Thickness *ft.*	Depth *ft.*
Quarternary Alluvium		
Sandy clay	20	20
Sand and gravel	33	53
Kanawha Unit of Pottsville Age		
Sandstone, white	9	62
Shale, white	6	68
Sandstone, water (12 gpm. at 70 ft.)	3	71
Shale	5	76
Sandstone	14	90
Shale, well full of water at 95 ft.	5	95
Sandstone	9	104
Shale	14	118
Limestone (?)	14	132
Shale	12	144
Sandstone, water	26	170
Shale	30	200
Sandstone	7	207
Coal	1	208
Limestone (?)	9	217

* Log of well at Coyle and Richardson Dept. Store, Charleston.

the Kanawha unit of Pottsville age, but the eastern section of the city is underlain by similar rocks of Allegheny age. The strata dip about 75 to 100 ft. to a mile as they roll in a northwesterly direction.

These strata have been divided into many named and unnamed units where they crop out at the surface (4), but commonly the logs of water wells in Charleston contain too little information to permit identification or correlation of the minor stratigraphic units. The log given in Table 2 illustrates the rock encountered in drilling.

Most of the wells in Charleston obtain water from these consolidated sandstones at depths averaging about 160 ft., and a few wells have encountered significant quantities of potable water more than 225 ft. below the surface. The yield of these wells ranges from less than 15 to more than 250 gpm. The thickness and permeability of the water-bearing formations vary somewhat, but some differences in yield also seem to be due to differences in depth and method of development of the wells.

The principal aquifer at Charleston cannot be correlated positively with named units, but the water-bearing sandstones are tentatively considered

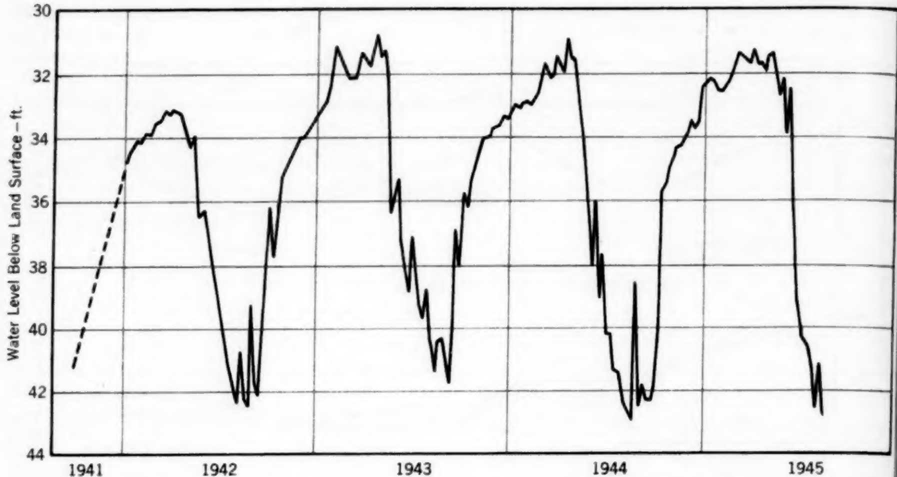

Fig. 3. Fluctuations in Water Level at an Observation Well

to be near the horizon of the Coalburg sandstone. In South Charleston, wells obtain water from the Mahoning sandstone of Conemaugh age, and in Nitro small quantities of water are pumped from another shallow sandstone in the Conemaugh.

The water in the bedrock formations is under moderate artesian pressure, so that it rises in wells above the level of the water-bearing stratum, and probably above the water table in the terrace alluvium. Data are not available now to indicate the general trend of water levels in wells throughout the city. The water level in a well at 800 Pennsylvania Ave. was 2.03 ft. higher in April 1943 than in October 1942. Weekly measurements of water level in an unused well near the corner of Lee and Dickenson Sts., shown in Fig. 3, indicate that increases in the withdrawal of water during the summer cause a large decline of the water level in these months, but that the levels recover during the winter and spring. Minor changes are probably due to the intermittent operation of pumps on nearby wells. The water level in this well was at its lowest observed stage on record in early August 1945. Operators of wells throughout the city report that water levels have declined appreciably since about 1939, but records of reliable measurements of water level are not available.

Quality of the Water

Most of the wells in Charleston yield water that is high in iron and rather corrosive. The water is clear as it comes from the wells, but the iron oxidizes rapidly to a flocculent reddish brown precipitate when exposed to the air. Since the waters are corrosive, a considerable proportion of the iron probably is dissolved from the casing and pipes as the water is withdrawn.

The character of water from representative wells is shown in Fig. 4. Water from the terrace alluvium is rather soft and generally contains less than 100 ppm. of dissolved solids. The relatively large percentages of sodium suggest that there may be some migration of water from the deeper bedrock aquifers through wells into the unconsolidated sand and gravel. This

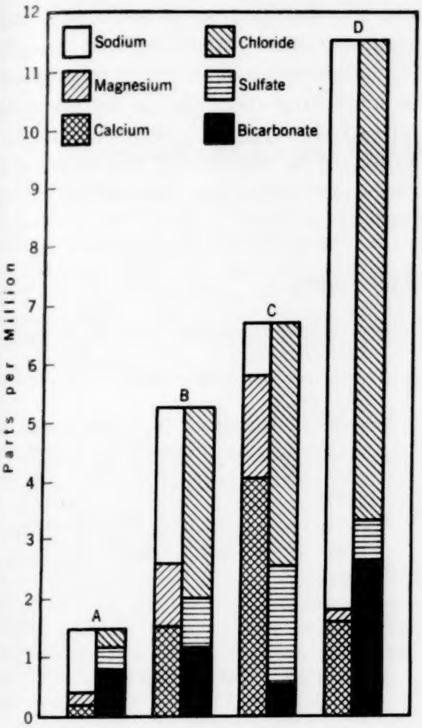

Fig. 4. Analyses of Water From Charleston Wells

Well A, analyzed by the U.S. Geological Survey, is drilled in terrace alluvium at the Conlon Baking Co. in Charleston. Well B, also analyzed by the U.S. Geological Survey, is in bedrock sandstone at the Daniel Boone Hotel. Well C is in bedrock sandstone at the Conlon Baking Co. and was analyzed by the West Virginia Geological Survey. Moderately deep bedrock sandstone at the S. S. Kresge Co. store provides the water for Well D, analyzed by T. A. Bordadoile of Charleston.

water from the terrace deposits is similar in general to the water in the public water supply system at Charleston. The water in the deeper aquifers, as shown in Fig. 4, contains appreciably more dissolved solids. The quantities of sodium and chloride increase rapidly with depth, so that the aquifers more than 230 ft. deep contain water that is a moderately concentrated brine.

Temperatures of ground water in Charleston range from about 55 to 62° F.

Utilization of Ground Water

Ground water in Charleston is used extensively by industrial and commercial establishments for the cooling of condensers in air-conditioning systems, cooling in dairy-product processing, general cooling in ice and packing plants, cooling machinery, boiler feed and laundering. Although the temperature is somewhat higher than is desirable for efficient cooling, the low cost and relatively constant temperature result in material savings to the users. The water must be treated before use in laundries or boilers and, commonly, treatment is desirable for waters used in some cooling processes.

Available records indicate that about 1 mgd. of ground water is used in the city of Charleston during the winter and more than twice as much in the summer months. The pumpage within ½ mile of the post office comprises more than half of this withdrawal in summer. The total known pumpage in the city, therefore, is about 535 mil.gal. per year. Records for the surrounding industrialized area are relatively incomplete, but indicate that at least an additional 345 mil.gal. of ground water are used yearly at industrial plants.

Conclusions

About 535 mil.gal. of water a year are obtained from wells in Charleston and 345 mil.gal. a year are obtained in the surrounding industrial area. This ground-water supply constitutes a valuable natural resource. The use of ground water has depleted available supplies in certain sections and has adversely affected some of the existing installations. Additional supplies of

water, however, can be obtained from wells in some parts of the area without seriously affecting the existing wells.

The overdevelopment has occurred in the bedrock aquifers in some parts of Charleston. On the other hand, the alluvial deposits do not seem to have been developed fully as yet. Therefore, additional studies are to be made by the state and federal geological surveys to obtain data on the quantity of water that is currently withdrawn from wells in each aquifer in different parts of the city and on the corresponding water-level fluctuations. Also information on the quality of the water in each aquifer, the methods of well construction best adapted to local conditions and the distribution of wells that will minimize interference will be sought. Consideration will be given to methods of increasing recharge, as by pumping pure cold water into wells during the winter or by increasing the amount of water percolating into the aquifers from the city.

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Abstracts of Water Works Literature

Key: In the reference to the publication in which the abstracted article appears, **34:** 412 (Mar. '42) indicates volume 34, page 412, issue dated March 1942. If the publication is paged by the issue, **34:** 3: 56 (Mar. '42) indicates volume 34, number 3, page 56, issue dated March 1942. Initials following an abstract indicate reproduction, by permission, from periodicals, as follows: *B.H.—Bulletin of Hygiene* (*British*); *C.A.—Chemical Abstracts; P.H.E.A.—Public Health Engineering Abstracts; W.P.R.—Water Pollution Research* (*British*); *I.M.—Institute of Metals* (*British*).

HEALTH AND HYGIENE

A Report on a Typhoid Fever Epidemic at Manteno State Hospital in 1939. Ill. Dept. of Pub. Health ('45). Singular court decision cleared official of responsibility for severe typhoid epidemic in '39 at Manteno (Ill.) State Hospital for Insane, involving 453 cases with 60 deaths. State Dept. of Pub. Health (DPH) acted to det. how typhoid transmitted at hospital, and set up control measures to prevent spread. *Administration and operation:* On July 1, hospital had 768 employees and 5,384 patients, ratio 1:7, under mgr. directly responsible to Director, Dept. of Pub. Welfare (DPW). At 72, director was "not a doctor, bacteriologist, nor any kind of scientist." Under him, clinical director of hospital med. dept. supervised 20 staff physicians and lab. technicians. Nursing dept. directed by trained nurse assisted by untrained lay supervisors who issued orders to registered nurses and contacted staff physicians. Lay attendants also responsible for nursing care. Dietary dept., under dietitian, 85% operated by dirty patients. Daily, 100 gal. raw milk from institution herd used only for cooking while main supply received from outside pasteurization plants. Except for few hospital cases, 19,815 meals prepd. daily in extremely dirty and foul-smelling central kitchen, infested with flies, as were dining rooms. Cooking utensils and meat-block cleaned with used floor-scrubbing brooms while most dishes unwashed or "dried" with meager supply of towels after 12-sec. exposure in dish-washer, if at all. None sterilized. Food prepd. in central kitchen, transported by trucks in unclean aluminum containers to ward kitchens, and dispensed. Vegetables prepd. in unclean, chipped bathtubs in peel-room crowded with unclean patients, often with floor flooded by stopped drains; then subjected to flies, dust, dirt and other contamn. when trucked 500′ around bldg. to central kitchen. Dishes used by typhoid patients returned to hospital kitchen unsterilized for general processing. When not dumped on ground, garbage collected in horse-drawn leaky box from unwashed open cans. Rubbish collection and disposal comparable. Rodents, flies and cockroaches, latter often found in patients' beds, flourished unrestrained. Flies overran hospital including operating and post-mortem rooms. With only 14 workers, master mechanic supervised water supply, sewage, garbage and rubbish disposal, insect and rodent control, power plant and grounds. *Water Supply:* Consisted of four partly-cased, drilled wells, only few hundred ft. apart in greatly cracked and creviced Niagara limestone covered with glacial drift from 0′ to 30′, dischg. to concrete ground-storage reservoir and pumped to distr. system on which pressure maintd. by elevated tank. During month before epidemic all water obtained from 225′ well No. 4 with 14″ casing 19′ from surface into limestone only 1′. Thus, glacial drift only 18′ thick and all water developed from creviced limestone. Static water level 26′ below ground surface during epidemic. Immediately following installation of water supply in '31, without previously submitting plans and specifications to DPH for review, institution officials notified that water samples therefrom showed contamn. and advised to boil the water and install chlorination. Requested by DPH, State Geol. Survey reported that proximity of limestone to present

surface dangerous for obtaining water; sewers and other constructions therein makes imperative that water from this well be watched constantly and perhaps treated with Cl_2 constantly. Managing officer and Director, DPW, so notified. Similar warnings given in '35 and '37. From '31 to Aug. '39, DPH tested approx. 26 water samples a yr. with avg. 27.3% portions pos. Reports of these results always sent promptly to above titled officers but disregarded because "there were more important things to take care of." After epidemic started, max. coliform indices from wells No. 4 and 1 were 70,000 and 240,000 per 100 ml. respectively. *Sewage:* Independent storm and some sanitary sewers of vitrified clay with cemented joints laid in creviced limestone. Sewage from 6,200 persons dischgd. to activated sludge sewage-treatment plant designed in '31 for 3,000. Resulting eff. unsatisfactory. *Sewer Leakage:* Temporary sanitary sewer stoppage occurred shortly before Aug. 22. Investigation showed domestic sewage flowing in parallel storm sewer 10′ to 15′ away. Both sewers in creviced limestone. In 15 min., fluorescein dye from nearby flushed toilets appeared in this sewer, then in parallel storm sewer 15 min. later. At same time storm sewer system on north side of institution practically dry while south side one carried > 5″ genuine domestic sewage. Signif. increase in chloride content of raw well water resulted from adding salt to sewers at various points. Thus tests proved sewage leakage from sanitary sewers into creviced limestone and thence into wells supplying water to institution. Until Aug. 19 this water pumped directly into distr. system without chlorination. Further proof in the definite isolation of *Eberthella typhosa* from 6 qt. raw water from well No. 4 by experienced bacteriologist between Aug. 22 and 31. Crowning evidence was bacteriophage typing of cultures from many fecal and urine specimens of typhoid patients and carriers, showing many phage types represented in epidemic. This suggested causative organisms transmitted to patients from many individual sources rather than by direct contact with single patient or carrier. "From this accumulated evidence it would seem likely that water supply was receiving direct contamn. from the institution sewer system." Control measures adopted immediately by DPH were: More stringent isolation of typhoid patients, better food-handling methods, proper disposal of typhoid wastes, improved methods of handling linen, insect and rodent control, adequate water and sewage chlorination, immunization and a carrier-finding survey. Managing officer, dietitian and Director of DPW indicted but charges against first two eventually dismissed while Director of DPW brought to trial in circuit ct. Because jury could not decide, defendant and state left decision up to judge who found defendant guilty of omission of duty as charged. This decision reversed by State Supreme Ct. with amazing 3500-word opinion climaxed by following: "It was not proved and we cannot assume that any typhoid bacillus was ever found in the drinking water. It was not proved and we cannot assume that there was ever any leak or defect in the system of plumbing and sewage disposal, or that any such contamn. entered either of the wells."—*Ralph E. Noble.*

Water-Borne Outbreak of Gastro-enteritis in an Industry. LEO LOUIS. Ind. State Bd. of Health Bul. **49**: 35 (Feb. '46). Serious water-borne gastro-enteritis outbreak occurred among 30 of 100 indus. employees in northern Ind. from Jan. 19–23, '46. Apparently epidemic caused by cross-connection in plant. Early Dec. freeze caused shutdown of creek pump drawing pold. water from point below sewer outlets in adjacent creek thus leaving town water sole supply for indus. use, with pressure, however, insufficient to flush toilets because of small-sized service main. Small well supplied single drinking fountain in factory. Two check valves installed near meter pit to prevent backflow of river water into town water mains, although, ordinarily, 3 gate valves separated the 2 systems. Figure 1 shows the plant water supply. Because of shutdown, 2 protecting gate

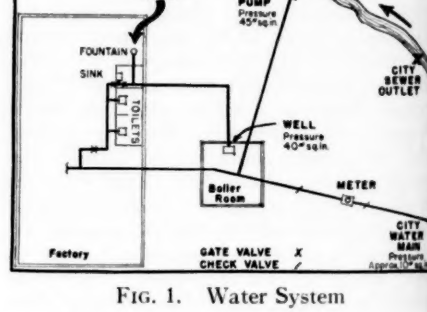

FIG. 1. Water System

valves opened Dec. 17, '45, to provide well water in toilet rooms. Uncertain whether third valve opened or left closed. From 10 A.M. Jan. 18 until 12:30 P.M. Jan. 19, creek pump operated with 45 psi. while well pump set to shut off at 40 psi., regulated by small pressure tank. Probable that operation of flush valve toilets reduced pressure in drinking water line. Cases of extreme intestinal cramps, diarrhea, nausea and vomiting occurred from night of Jan. 19 to Jan. 23 and, among them, water only common food or drink consumed. Water samples collected Jan. 23 showed flushing with well and city water removed most of contamn., although some remained in drinking water line. Fountain and indus. line satisfactory on Jan. 24, indicating effective flushing.—*Ralph E. Noble.*

Cyanosis in Infants Caused by Nitrates in Well Water. H. H. Comly. J. Am. Med. Assn. **129**: 2: 112 (Sept. '45). Two examples of previously unrecognized condition of cyanosis in babies which, it is maintd., is due to high nitrate content of water consumed, described. Stated that condition not by any means rare in rural areas of Iowa. First case artificially fed baby 4 wk. old, with history of diarrhea and vomiting, who was admitted to hospital as emergency case of cyanosis with drowsiness. Oxygen treatment ineffective. Injection of 1.0% soln. of methylene blue (1.1 ml. for each kg. of body weight) under scalp restored child within 30 min. Baby sent home after week but readmitted twice with same signs and symptoms, which on second occasion extremely severe. Father of child had theory that well water used in making up artificial foods cause of condition. This theory considered. Child had had high normal value of methemoglobin in blood, and reports in literature of infantile methemoglobinemia caused by bismuth subnitrate suggested that nitrates in water might have been causative agent. Water found to contain amt. of nitrate equiv. to 1.0 g. of potassium nitrate per l., and bacteriologically was highly pold. Mechanism of causation of cyanosis accordingly believed to have been conversion of hemoglobin into methemoglobin by chem. action of nitrites, absorbed through damaged intestinal mucosa, after having been produced by action of bacteria in intestinal tract on nitrate in water used in making up artificial foods. Seems advisable to recommend that water used in infant feeding contain no more than 10 or at most 20 ppm. nitrate.— *B.H.*

Unsafe Water Wells. L. T. Watry. Wis. Qtrly. Bul. **7**: 12 (Oct.–Dec. '45). Series of questions and answers brings out elementary principles of unsafe wells, coliform test and chlorination methods according to well type. Dosage table follows:

Well Type	Chlorinated Lime, *oz.*	Water, *gal.*
Driven point	1	5
Tubular	1	10
Dug, according to size	1–5	10–50
Bored, according to depth	1–2	10–20
Drilled, according to size and depth	1–5	10–50

Six measuring teaspoonfuls = 1 oz. chlorinated lime.

After lime soln. settled, add clear liq. to wells thus: (1) *Driven point and Tubular:* Remove pumping equip. (including check-valve from tubular well). Pour in rapidly. (2) *Dug:* Uncover well, pour in soln. carefully to flow over all curbing surface above water line. Agitate water in well thoroughly. (3) *Bored or Drilled:* Raise pump or remove well-cap and pour in soln. rapidly. (4) *Flowing:* Prepare container that will enter well and sink to bottom. Attach suitable line. Place chlorinated lime into container, lower into well, moving up and down near bottom until Cl_2 released. Remove container. Leave lime soln. in well 3 hr. or more, then flush by pumping. During first 30 to 40 min. return treated water to wells of dug, bored or drilled type to insure thorough treatment of all surfaces above water line. When Cl_2 no longer perceptible by taste or smell, collect sample directly into sterile bottle (usually supplied by testing lab.) and forward same to nearest state lab. for anal. Disinfection elimns. only poln. currently existing. Structurally unprotected well may immediately be rendered unsafe if pollutional matter enters. *Emergency sterilization of drinking water.* Drinking water made safe for domestic use by adding small quants. of chloride of lime. Mix one measuring teaspoonful thoroughly with 1 pt. water, let settle, then use clear part as stock Cl_2 soln., preserving it in tightly stoppered dark bottle in cool place. One teaspoonful will sterilize 20 gal. of water; 18 drops for 1 gal., and 5 drops for 1 qt. Let stand at least 15 min. after dosing. Slight Cl_2 taste should be noticeable. Also water may be sterilized

by adding 6 drops of tincture of iodine to 1 gal., letting stand ½ hr. before use.—*Ralph E. Noble.*

Progress Being Made by Federal or State Authorities on Regulations Pertaining to Railway Sanitation. H. W. VAN HOVENBERG ET AL. Am. Ry. Eng. Assn. Bul. *No.* 455: 79 (Nov. '45). Reference made to revision of U.S.P.H.S. Drinking Water Stds. and to Sanitation Manual for Land and Air Conveyances in Interstate Traffic. Revised section on hydrants for furnishing drinking water indicates that severe restrictions will be placed on railroads in installation of new watering hydrants.—*R. C. Bardwell.*

Warning—Emergency Operation of Public Water Supplies. ANON. N. J. Pub. Health News. **27**: 347 (Oct. '45). All water purveyors, local boards of health, and licensed water treatment plant operators notified Aug. 20, '45, by Director of N. J. Dept. of Pub. Health, about application of law R.S. 58:11–7 to various types of emergency water supply operation as follows: Any person furnishing potable water and obliged for any reason to make any temporary or permanent change in operation of plant or manner of furnishing such water, tending temporarily or permanently to deteriorate potable qual. by pumping untreated water directly into reservoirs or supply mains, when ordinary supply usually subjected to purif. treatment, or by any other change in such supply tending to force pold. waters into distr. pipes, shall beforehand, or within 6 hr. after emergency requiring immediate changes, notify local board of health and, by telegraph or telephone, state dept. of health, of character and estd. duration of such change. Explained that neglect or delay in notifying dept. of emergency operations caused by storms resulted in unnecessary delay in emergency treatment and restoration to normal. Prolonged threats to safety of potable supplies ensue. Remarks prompted by experience with '44 hurricane and '45 floods.—*Ralph E. Noble.*

How Does Your Community Measure Up to Sanitation? WARREN J. SCOTT. Conn. Health Bul. **60**: 27 (Feb. '46). Good community sanitation envisions: safe and adequate water supplies, sewage treatment, garbage-rubbish disposal, healthful housing (accompanied by insect and rodent control), safe food and safe recreational facilities. Sanitation varies with locality. Certain local advantages may include: Existing nearby sources of water for pub. supplies and large vols. of dilg. water near rivers or shores to aid sewage disposal; past planning of facilities; extent to which health agencies developed to meet supervisory and health educ. needs. Future sanitation level depends upon interest of avg. citizen in supporting local well-working depts. and health agencies. Striking reduction in cases and deaths from gastro-intestinal disorders in Conn. significantly related to safer water supplies. Some communities without pub. water systems have increased in pop. to point where community health demands them. State health dept. performs important function co-operating with local health agencies in bldg. inspections of water and sewage piping particularly to elim. cross-connections and other health hazards. Considerable no. of Conn. communities still unprovided with sewage treatment facilities. Some have plans in making or execution. Public now realizes poln. must be checked and indiscriminate dischg. of wastes into streams and shore waters controlled. Full-time health depts. should render good pub. service and can also aid by extending activities into housing field. Where part-time health service cannot afford satisfactory level of supervision and promotion of sanitation, permissible to form dist. health units whereby 2 or more communities benefit from full-time operating health agency.—*Ralph E. Noble*

Rusty Water and Mosquito Breeding. ANON. Nature (Br.) **154**: 714 ('44). Review of report by K. B. WILLIAMSON on **Investigation of Ferruginous Waters in Relation to the Breeding of Malaria-Carrying Mosquitoes.** Commonly observed that in tropics mosquito larvae not found in waters which contain rusty deposits or which are covered with film of pptd. iron. Suggested that this fact might be utilized in control of mosquito breeding. Author of report has examd. various types of rusty water in Great Britain and in Malaya. Iron occurs in water as colloidal ferric hydroxide, in true soln. as ionized salts of iron and as non-ionized org. complexes. It is colloidal iron which is unstable and gives rise to solid aggregates in form of ppts. or surface films. Ferruginous water defined as one which contains at least 2 ppm. iron in soln. or colloidal suspension. Rust-colored sediment may have been de-

posited at earlier date and is not evidence of water satd. with iron. Film of rust on surface, however, indication that there is unstable excess of iron in water. Iron may be derived from underground mineral sources such as pyrites, from vegetation rotted under anaerobic conditions in marshy soil contg. iron thus forming org. complexes with iron, or, to limited extent, from vegetation rotting in water. Ferruginous waters characterized by almost complete absence of water insects, helminths, crustacea and algae. Newly hatched larvae of *Anopheles maculipennis* unaffected by ferruginous waters when they were fed but they soon died of starvation when no food added. Therefore concluded that absence of mosquito larvae in rusty waters due to lack of food. No evidence that surface films contg. iron will suffocate larvae. —*W.P.R.*

Fluoride Removal From Drinking Water. Small Installations Using Virgin Bone Black. A. L. Burwell, L. C. Case & C. H. Goodnight. Okla. Geol. Survey, Circ. No. 25: 1 ('45). Summary of methods used for removing fluorides from drinking water given. Presence of fluoride in excess of 1.0 ppm. makes water unsafe and ground for rejection of water supply. Waters contg. fluorides in amts. above safe limit can be made safe by using virgin bone black, which was found to be superior to Ca phosphate in eff. and ease of regeneration. Also removes odor and taste. Map shows general distr. of known fluoride waters in Okla. and table lists water supplies in state contg. as much as 0.6 ppm. fluorides.—*C.A.*

Fluorine in Dental Public Health—A Symposium. Frederick S. McKay et al. N.Y. Inst. of Clinical Oral Pathology. 62 pp. ('45). Five papers in symposium trace history and development of theory of relationship of fluorine to enamel of teeth. McKay cites evidence from studies starting in Colorado Springs in '08, that excessive amts. of fluorine in drinking water during period of calcification of permanent teeth caused condition known as mottled enamel. Also reviews later recognition of beneficial effects of fluorine at level of 1 ppm. in reduction of dental caries. Dean gives results of studies of caries incidence in permanent teeth of children, in relation to fluorine content of drinking waters in 21 cities in Middle West, and presents striking evidence that fluorine concn. less than 1 ppm. in water supply leads to greatly increased caries incidence. Armstrong shows that fluorine content of enamel from carious teeth avgs. only 62% of that found in enamel of normal sound teeth, and Bibby gives results of topical applications of fluoride to surfaces of teeth showing that this method has promising possibilities when used with children. Final paper in symposium gives details of long-time controlled expt. that is under way in N.Y. State to det. efficacy, practicability and safety of reducing prevalence of dental caries by addn. of sodium fluoride to potable water supply of community.—*P.H.E.A.*

A Dental Study of School Children in Eleven Fluoride and Fluoride-Free Communities in Michigan. C. Ray Taylor. **34:** 8 (Jan. '46). Dental conditions in 410 school children from Mich. communities using waters contg. 0.1 to 1.8 ppm. F. Age of these continuous-residence children ranged from 4 to 18 yr. Deciduous teeth found in 283 children from fluoride areas and 286 from nonfluoride areas. Avg. "def" (total of all decayed, indicated for extraction and filled deciduous teeth) per child was 5.5 in first group, 3.18 in second. Permanent teeth possessed by 351 children in fluoride areas and 433 in non-fluoride areas. DMF (total of all decayed, missing and filled permanent teeth) per child was 2.21 in first and 5.71 in second groups. Percentage having permanent teeth without caries experience (no. decayed, missing or filled teeth) was 38.7 and 13.4 in same respective groups. Number of children with teeth in need of cleaning and those with gingival conditions (affected tissue around tooth) not significantly different in either area.—*Ralph E. Noble.*

Flourine as an Inhibitor of Dental Decay. John G. Frisch. Jour. Wis. State Dental Soc. (Nov.–Dec. '44). *Abstract,* Off. Bul. N. D. Water & Sew. Wks. Conf. **12**: 6 (Apr. '45). Mass control of dental decay impossible and impractical by diet alone. Any substance worthy as inhibitor of dental caries must be effective against high carbohydrate diet of avg. American. Well-balanced diet plus 1 ppm. F in drinking water supply will cut decay incidence drastically. Addn. of 1 ppm. F to fluorine-deficient water supplies is harmless, safe and feasible. Sensible procedure will largely solve pressing public health problem of dental caries. Duty of

medical and dental professions to point way. Subject interesting and fascinating. When public sufficiently informed and demand arises to thus fortify deficient munic. supplies, great service rendered humanity.—*Ralph E. Noble.*

Preventing Dental Caries by Fluoride Treatment of Water. M. STARR NICHOLS. Wis. Qtrly. Bul. **7**: 26 (Oct.–Dec. '45). Data from numerous sources by many scientists from communities using water contg. 1 to 2 ppm. F compared with ones using water of little or no F provides conclusions that: children drinking water of 1 ppm. F from infancy to adult life will experience about $\frac{1}{3}$ tooth decay (dental caries) of those drinking water contg. little or no F. One ppm. harms neither child nor adult pop. but ultimately produces gen. pop. in better health through developing sound teeth in child. Reasonable diets contg. adequate amts. of Ca, P, vitamins D and A, and fluorides in drinking water to 1 ppm., will greatly reduce tooth decay in adult life of present children. One ppm. F in drinking water will not prevent tooth decay in adult because protection depends upon incorporation during tooth-growth process and does not replace regular dental care.—*Ralph E. Noble.*

The Relationship of Fluorine Content, Hardness, and pH Values of Drinking Water and The Incidence of Dental Caries. T. OCKERSE. S. African Med. J. **18**: 255 ('44). In investigation into incidence of dental caries in South Africa, teeth of 78,563 school children examd. and hardness, pH value and fluorine content of samples of drinking water from dists. concerned detd. Results showed that incidence of dental caries related to F content in drinking water and to hardness and pH value of water. Incidence of dental caries low in dists. where F content in water greater than 1 ppm. and where water hard or had high pH value. In places where water fulfilled all three conditions, incidence of caries particularly low.—*W.P.R.*

Dental Caries in A High and Low Incidence Area in South Africa. A Study of Possible Contributory Factors With Special Reference to Diet. M. MALHERBE & T. OCKERSE. S. African J. Sci. **9**: 75 ('44). In investigation into causes of high incidence of dental caries in George area of South Africa compared with Williston area, special consideration given to diets of respective pops. Differences in geographical position, in climatic conditions, in geological structure, in vegetation, and in compn. of water supplies and soils in 2 areas summarized. Crops grown in George area analysed and found to have normal content of magnesium, high content of iron, and low contents of calcium and phosphorus. Fluorine content, total hardness and pH value of samples of drinking water from each area compared. Results of detns. of contents of calcium, magnesium, phosphorus and fluorine in teeth showed that enamel and dentine of teeth from persons living in George area contained less fluorine than those of teeth from persons in Williston area. Believed that diet in George area deficient in proteins, fats, calcium and necessary vitamins. Discussion of probable relation between solar radiation, vitamin D and calcification included.—*W.P.R.*

Domestic Water and Dental Caries. The Relation of Fluoride Domestic Waters to Permanent Tooth Eruption. E. M. SHORT. J. Dental Research. **23**: 247 ('44). Investigations made in 12 communities in Illinois and Colorado to det. whether time of eruption of permanent teeth of children affected by content of fluorine in public water supplies. At Colorado Springs, Colo., where content of fluorine in water supply 2.6 ppm., eruption of permanent teeth of 12-year-old children appeared to be delayed when children had for long periods been drinking public water supply. In dists. where supplies contained less than 2.0 ppm. fluorine, no significant difference in time of eruption of permanent teeth observed. Author considers that marked difference in incidence of dental caries among pops. using water supplies free from fluorine and among those using water supplies contg. up to 1.2 or 1.3 ppm. fluorine bears no relation to difference in times of eruption of permanent teeth.—*W.P.R.*

The Toxicity Thresholds of Various Substances Found in Industrial Wastes as Determined by the Use of Daphnia Magna. B. G. ANDERSON. Sewage Works J. **16**: 1156 ('44). Use of *Daphnia magna* as exptl. animal for detection of toxic substances in water discussed. Previous workers have shown that susceptibility of this organism varies with age. Expts. made by author to det. toxicity at 25°C. of 42 substances to individuals of *Daphnia magna* not more than

8 hr. old described. Series of dilns. of soln. of each chem. made; solns. dild. with Lake Erie water which had been centrifuged to remove most of suspended matter. Animals observed 15 min., 30 min., and 1, 2, 8, 16 and 32 hr. after their transfer to solns. and time at which they had become immobilized, i.e., at which they had ceased swimming, noted. Threshold concns. of 42 substances, that is highest concns. which just failed to immobilize animals under prolonged exposure, shown in a table; concns. given in molar concns. and in ppm. Substances tested include salts of aluminum, ammonium, barium, calcium, cobalt, copper, iron, potassium, sodium and zinc, a number of inorganic and org. acids, acetone, aniline, phenol and three alcs. In case of aluminum salts threshold concns. inversely proportional to number of aluminum atoms in molecule. Sulfuric acid twice as toxic as nitric or hydrochloric acid. Tartaric acid as toxic as sulfuric acid; oxalic acid slightly less toxic than sulfuric acid. In case of methyl, ethyl and isoamyl alcs. toxicity increased with increase in number of carbon atoms in molecule.—*W.P.R.*

The Role of pH in Determining the Toxicity of Ammonium Compounds. W. A. Chipman. Thesis (Ph.D.) Univ. of Missouri, '34; Microfilm Abstr., **5**: 2:3 ('44). Expts. made to det. effect of pH value on toxicity of ammonium salts to aquatic animals described. In soln., ammonium salts dissociate chiefly into ammonium ions and acid ions, but small amts. of the free acid and of ammonium hydroxide also formed; the hydroxide dissociates further, forming water and ammonia gas. Pharmacological action of solns. of ammonium salts due both to ammonium ions and to free ammonia. Expts. showed that alk. ammonium salts much more toxic to goldfish than were acid salts; e.g., 0.005 *N* soln. of ammonium carbonate in distd. water killed goldfish in 3.58 hr. on avg., although 0.1 *N* soln. of ammonium chloride did not kill fish within 12 hr. Toxicity of solns. of ammonium chloride much greater when prepd. with tap water than when prepd. with distd. water. 0.05 *N* soln. of ammonium chloride prepd. with distd. water not toxic to goldfish, but became toxic when sodium hydroxide, sodium carbonate or disodium hydrogen phosphate added; increase in toxicity proportional to decrease in free hydrogen ions. Addn. of sodium dihydrogen phosphate which increased acidity of soln. did not make it toxic. Direct relation between pH value and toxicity of solns. of ammonium chloride to fish, amphipods and cladocerans demonstrated. This variation in toxicity apparently directly associated with dissociation of ammonium salt, as results were same whatever salt used to regulate pH value. Author concludes that toxicity of ammonium salts directly related to amt. of undissociated ammonia in soln.; this is detd. by concn. of free hydrogen ions present.—*W.P.R.*

Greenhouse Studies of the Toxicities of Oklahoma Salt-Contaminated Waters. R. E. Wall & F. B. Cross. Okla. Agric. Expt. Sta. Tech. Bul. No. T-20, 32 pp. ('43). In expts. to det. toxicities to various plants of salts present in Oklahoma salt waters, found that 500-ppm. concn. of salts injurious to more susceptible plants and that when concn. reached 1000 ppm. growth of all plants impaired, particularly if conditions of soil, light and temp. unfavorable. Species of plants and types of soil which give satisfactory results with water contg. high concn. of salts, corresponding conditions required for watering, and conditions of temp. and humidity discussed. Water which contains less than 200 ppm. salts and little or no alk. excellent for use in greenhouses. No effective chem. method devised for reducing toxicity of salt water.—*W.P.R.*

CHEMICAL ANALYSIS

Separation and Determination of Very Small Amounts of Al and Fe in Water. N. Strafford & F. F. Wyatt. Analyst **68**: 319 ('43). Colloidal $Al(OH)_3$ and org. matter, by coating granules of carbonaceous zeolite in softened water, reduced working exchange capac. of water softening plant to about 60% of its capac. Expts. showed that Al, Fe, org. matter and turbidity could be pptd. by coagulation with filter alum at pH of 5.8–6.4. During exptl. work desired to det. accurately about 1 ppm. of Al in feed water and 0.005–0.1 ppm. Al in final treated water. Detn. based on either aurintricarboxylic acid or

hematoxylin colorimetric procedure but presence of Fe interferes. Ions of this element react with both the above reagents. Fe best removed by extraction of $Fe(CNS)_3$ with amyl alc. and ether in presence of a little HCl. After this extraction Al detd. with hematoxylin at pH 7.5 or with aurintricarboxylic acid at pH 7.5. Procedures modified so that 0.1–0.5 ppm. of Al can be detd. within 0.01 ppm. Modified hematoxylin method rather more convenient and slightly more sensitive than modified aurintricarboxylate method. Full details given with slight changes according to quants. of Fe and Al present.—*C.A.*

Detection of Copper Ion. N. A. Tananaev & V. N. Podtschainova. Zavod. Lab. (U.S.R.R.) **9**: 168 ('40). By introduction of cadmium sulfide into soln. contg. copper, 0.5 mg. of copper in 1 l. can be detected. Cadmium sulfide will darken if concn. of copper in soln. above certain value, but below this value ppt. treated with dil. hydrochloric acid; copper sulfide then detected in residue. If bismuth, silver or mercury present in soln., pptd. sulfides dissolved by means of aqua regia; on addn. of concd. ammonia blue color appears if concn. of copper greater than 10 mg./l.—*W.P.R.*

Application of the Xanthydrol-KOH-Pyridine Method to the Determination of 2,2-bis (*p*-Chlorophenyl)-1,1,1-Trichloroethane (DDT) in Water. Julio C. Castillo & Henry A. Stiff Jr. Military Surgeon, **97**: 500 ('45). Red color produced when DDT heated in anhyd. pyridine soln. contg. xanthydrol and solid KOH adapted for detg. this compd. in water. Procedure: 200 ml. water contg. 0.1–4.0 ppm. DDT, strongly acidulated (litmus) with glacial AcOH extended with 100 ml. Et_2O; 50 and 25 ml. aliquots of Et_2O layer carefully evapd. to residue without AcOH odor. Reisdues separately dissolved in 5 ml. Et_2O (with 2 washings of 5 ml. Et_2O each), and transferred to test tubes for evapn. of Et_2O at 120° (oil bath). Reagent prepd. fresh daily from 50 ml. of 0.2% soln. of xanthydrol in colorless pyridine refluxed in glass while 25 dry pellets of KOH added through condenser. Flask swirled at 15–20 sec. intervals until supernatant dark green. Then hot soln. free from undissolved KOH transferred to dry Pyrex flask. Supernatant immediately loses its green color. Ready for use. Two ml. reagent added to tubes contg. aliquot residues and to blank tube; tubes heated at 120° for 8 min., then in cold water 1 min., when 4 ml. pyridine added, contents mixed, and transferred to dry photoelec. colorimeter cells. Green screen (520 mμ) used, blank set at 0 and readings taken on aliquots within 10 min. after removal from oil bath. Amt. of DDT calcd. from std. color curve prepd. from known amts. of DDT. In tap water, deeply colored turbid water and sea water contg. 0.1–1.0 ppm. DDT from aq. triton-xylene emulsions, and kerosene emulsions contg. 5% DDT (mosquito larvicides) 75–110% of added DDT recovered by this method.—*C.A.*

Determination of Fluorides in Water by Means of a Photoelectric Colorimeter. Osman James Walker & Gordon Clements Gainer. Can. J. Research. **23B**: 275 ('45). Method based on bleaching of lake formed from zirconyl salt and Na alizarinsulfonate by action of fluoride ion. Direct-reading photoelec. colorimeter based on that developed by Evelyn used but differs in that light passes vertically through long absorption cell rather than horizontally through absorption cell consisting of large test tube. Galvanometer scale calibrated against solns. contg. known amts. of fluoride. Results agree with those of corresponding visual method.—*C.A.*

The Determination of Fluorine as Lead Chlorofluoride. W. Kapfenberger. Aluminium (Ger.) **24**: 428 ('42); Chem. Zentr. (Ger.) **1**: 2013 ('43). In detn. of fluorine as lead chlorofluoride, lead acetate cannot be used as pptg. agent because of formation of basic lead chloride, and because there is danger of pptn. of lead fluoride with lead chlorofluoride. Method described in which lead chloride used as pptg. agent. After filtration ppt. washed first with satd. soln. of lead chlorofluoride and finally with as little water as possible; then dissolved in nitric acid. Chloride ion pptd. by adding soln. of silver nitrate. Mixt. then filtered and aliquot part of filtrate titrated with ammonium thiocyanate. This method particularly suitable for anal. of aluminum fluoride and cryolite.—*W.P.R.*

Rapid Determination of Magnesium in Waters by *o*-Hydroxyquinoline. R. Buydens. J. Pharm. Belg. (Belg.) **1**: 3 ('42). In detg. magnesium in water using *o*-hy-

droxyquinoline, 1 ml. of 20% potassium oxalate added to 50 ml. of sample of water, which is then filtered after standing for 5 min. Filter should not be washed. To filtrate 2 ml. of 2% alcoholic soln. of *o*-hydroxyquinoline and 2 ml. of concd. ammonia added. Soln. heated to boiling, allowed to stand for 5 min., and filtered; ppt. washed with 2.5% soln. of ammonia. Ppt. dissolved in 10% hydrochloric acid and soln. titrated with 0.1N soln. of bromine until slight excess of bromine present; indigo carmine or methyl red used as indicator. 20% soln. of potassium iodide added and liberated iodine titrated with 0.01N thiosulfate, starch soln. being used as indicator. When 50 ml. sample used, 1 ml. of 0.1N bromine equiv. to 10.08 mg. of magnesium oxide per l.—*W.P.R.*

The Accuracy of Measurements of Dissolved Oxygen in Water. I. M. Sillars & R. S. Silver. J. Soc. Chem. Ind. (Br.) **63**: 177 ('44). Studies made to exam. possibility of accurate detn. of D.O. in water at concns. less than 0.01 ml./l. In Winkler test errors introduced by personal variations in assessment of color at end-point, by introduction of oxygen with reagents and by fact that color not visible with starch until iodine in soln. equiv. to about 0.03 ml. of oxygen per l. Statistical basis of studies discusssed. Std. deviations of results obtained with samples of 250, 500, and 1000 ml. compared; greatest accuracy obtained with 500-ml. samples and further tests therefore made with samples of this size. Ordinary 500-ml. stoppered bottles used for collection of samples; in view of slow rate of absorption of oxygen by water, special air-tight containers considered unnecessary. "Dead-stop" method of electrometric titration used. Platinum electrodes 6 cm. apart, and potential of 150 m. applied. Std. sodium thiosulfate added in sufficient quant. to polarize electrodes and soln. titrated with std. soln. of iodine until first permanent deflection of galvanometer obtained. Ultimately addn. of 1 drop of 0.002 *N* iodine gave permanent deflection. When Glasgow tap water or distd. water used, found that iodine equiv. to 0.005 ml. of oxygen per l. must be present before deflection of galvanometer obtained when std. iodine soln. was of order of 0.002 *N*. With impure water this factor would be much greater because of absorption of iodine by org. matter or reducing substances. In order to minimize error due to oxygen introduced with reagents, pure reagents, frequently renewed, used and introduced below surface of samples from pipettes. Water to be analyzed allowed to flow at equal rates into 4 bottles simultaneously; tubes delivering water reached to bases of bottles. Water allowed to overflow from bottles for 5 min.; rate of flow such that bottles filled approx. 7 times. Glass stoppers wetted with water to be analyzed and inserted in bottles, care being taken that no visible air bubbles entrapped. Samples collected at 65°F. Std. Winkler reagents employed. After addn. of manganous chloride, potassium hydroxide and potassium iodide, the sample inverted 3 or 4 times and allowed to settle for 5 min. Acid then added and sample transferred to 1-l. beaker for titration. During titration liquid stirred. Usual amts. of reagents added to 2 of 4 samples; double quants. of reagents added to other 2 samples. Titration carried out with each sample by adding slight excess of std. sodium thiosulfate (about 0.004*N*) and titrating with std. iodine soln. (about 0.002*N*). Blank test then performed by adding more thiosulfate and titrating with iodine. Difference between 2 values for each sample represents iodine originally present in soln., expressed in terms of std. iodine. By taking 4 possible combinations of result obtained with pair of samples contg. normal amts. of reagents and with pair of samples contg. double amts. of reagents, 4 values can be obtained for oxygen initially present in samples. Mean value and std. deviation calcd. from these values. Using this technic in series of tests, std. deviation between 0.001 and 0.0025 ml./l. and independent of mean oxygen value of sample. Taking higher of these 2 values, "significance range" of procedure, i.e., min. quant. which can be significantly differentiated from zero, is 0.004 ml. of oxygen per l. This significance range compared with that obtained by using starch as indicator, same procedure being carried out before titration. With starch, when avg. concn. of oxygen above 0.04 ml./l., std. deviation was between 0.006 and 0.010 ml./l. and significance range was 0.016 ml./l. Series of tests made to compare values obtained with theoretical values. Air bubbled through tap water so that water satd. with oxygen. At 65°F. theoretical concn. of oxygen in such water is 6.55 ml./l.; this figure confirmed experimentally. Different amts. of this water added to de-aerated water and concns. of oxygen in mixtures calcd. and detd. experimentally. Concns. of oxygen in sam-

ples between 0.01 and 0.14 ml./l. None of detd. values deviated significantly from theoretical values.—*W.P.R.*

Colorimetric Determination of Some Phenols and Naphthols. B. N. Afanes'ev. Khim.-Farm. Prom. (U.S.S.R.) **7**: 18 ('44). In method for colorimetric detn. of phenols and naphthols, based on color formed in reaction of chloramine-T with phenols in aqueous soln., 50 ml. of 10% soln. of chloramine-T added to soln. contg. 0.1–2.0 g. of phenol; resulting soln. then made up to 1 l. with distd. water. After 2 hr., color compared with that of std. prepared from same phenol. Results of detns. of resorcinol, hydroquinone, pyrogallol, *p*-amylphenol, phloroglucinol and α- and β-naphthol given. Phenol not detd. by this method since it gives no color with chloramine-T. Expts. on detn. of mixts. of phenols did not give satisfactory results.—*W.P.R.*

Determination of Sulfates in Potable Waters. A. Nau. Bul. trav. soc. pharm. Bordeaux. (Fr.) **83**: 9 ('45). Add 2 g. of powd. $BaCrO_4$ and 2 ml. of reagent HCl to 200 ml. of sample. Boil for 2 min., add NH_3 until odor definitely perceptible, cool, transfer to 250-ml. volumetric flask and make up to 250 ml. with distd. water, washing flask carefully with added water. Filter, passing filtrate through filter until liquid perfectly clear. To 125 ml. of filtrate add 1 g. of iodate-free KI and 10 ml of H_2SO_4 (1:5). Titrate liberated I with 0.05 *N* thiosulfate. Number of ml. of thiosulfate consumed $\times$ 0.00163 gives sulfate per l. of water expressed as H_2SO_4.—*C.A.*

Studies on the Chemical Preservation of Water Samples. Claude E. ZoBell & Barbara Fay Brown. J. Marine Research (Sears Foundation) **5**: 179 ('44). Toluene, cresols, thymol, CS_2 and $(C_2H_5)_2O$ not dependable preservatives. $CHCl_3$ (0.5%), HCOH (0.25%), PhOH (0.5%) or mixture of PhOH (0.5%) and $CHCl_3$ (0.5%) can be used. Some micro-organisms will live in up to 1% PhOH, but not in mixture of $CHCl_3$ and PhOH. Acidifying water to pH 1.5 prevents activity of micro-organisms ordinarily found in water.—*C.A.*

New Facts Concerning the Piccardi Phenomenon. A. Manfredi. Gazz. chim. ital. (It.) **72**: 529 ('42). Known that when distd. water, or water from spring maintd. for certain time in contact with polarized dielectric, it acquires new properties by what is known as Piccardi activation. Activation may be of 2 types known as T and R. T activation causes calcium carbonate to be pptd. at lower temp. than that required with normal water, whereas R activation delays this pptn. to higher temp. As result of expts. unit for measuring degree of activation established. Cryst. forms of calcium carbonate deposited from normal water and from 2 types of activated water illustrated.—*W.P.R.*

The Chemical Analysis of Water. Gordon Carter. Wtr. & Wtr. Eng. (Br.) **47**: 390 (Sept. '44). Today chemist welcomed by engr. who realizes that chemist is really of some use. To nearly end of last century water examiner based his opinion as to suitability of water supply on results of chem. anal. alone. Chem. anal. has lost much of its immediate importance. Value of chem. anal. to engr. has taken on different but no smaller significance. This usefulness becomes evident in sterilization, softening, coagulation and numerous other problems of chem. nature. Sterilization now almost synonymous with chlorination. At first chem. anal. invoked in humble way. It supplied information about variable percentage of available Cl_2 in reagent employed. Chem. research showed that tastes due to chlorine are of two kinds. More recent work has shown that free residual will exert considerable influence on sterilization practice. Interpretation of results of chem. anal. can assist engr. in deciding on policy of chlorination at existing works or in design of new works. Chem. anal. essential in making decision as to which process of water softening to adopt. In considering usefulness of chem. anal. to engr. in design and operation of water softening plant, necessary to consider separately two types of plant. Once base-exchange plant installed, calls for chem. anal. less exacting than with lime-softening plant. In clarification of water by alum or other coagulant, found that waters vary in their requirements and chem. data help to decide choice of coagulant, auxiliary chems., methods and points of application, doses required and removal of added substances from water. Engrs. who handle ferruginous waters find that they have to remove iron. Some waters are plumbosolvent. Method of correction has to be detd. Engr. will be consulted by certain consumers who need water

of std. not fulfilled by his supply. Consumers have become more water conscious. Not invariably true that detection of poln. by chemistry has in recent years given place to bacteriology. In one case poln. recognized by higher figure for free and albuminoid ammonia. Confirmation by bact. tests not forthcoming until next day. Poln. of water supplies by chem. poisons not normally matter calling for close attention by chemist. In times of war, however, one has to be prepd. for such eventuality. Instances have occurred where sources of poln. of underground water have been cleared up by addn. of harmless chem. substance. Chemist can often save time and labor by decision whether or not "leakage" samples are or are not from main owned by water works. Control of boiler water treatment necessitates chem. tests on feed water and blowdown —*H. E. Babbitt.*

pH Determinations of Water in Contact With Stones. H. G. Williams. J. Soc. Chem. Ind. (Br.) **62**: 209 ('43). Expts. made to det. pH values of water left in contact with several types of stone used for making roads. Measurements of pH value made with glass electrodes. After prelim. expts. had been made with containers of several different materials, decided to use pyrex glass for main series of expts. because this had least effect on pH value of water. When pieces of pyrex glass shaken in pyrex bottle contg. distd. water with initial pH value of about 5, pH value increased to 6.9 in ½ hr. and to 8 in 3 hr. When several types of stone shaken in pyrex bottles, considerable increases in pH value observed after ½ hr.; did not increase appreciably on further shaking. Higher pH values obtained with granite than with limestone; appeared to be due to greater abrasion of glass by granite. In subsequent expts., therefore, bottles shaken by hand to avoid abrasion as much as possible. In one series of expts., limestone, granite, quartzite and gravel placed in contact with distd. water and pH values of water detd. every few days. With one exception stones all ¼ to ½″ in size; 200 g. of stone placed in 150 ml. of water. pH of all waters increased but only slightly with Ham R. gravel. Highest pH obtained in water in contact with limestone. pH of water in contact with limestone decreased after first day, whereas pH of water in contact with other stones remained approx. const. In another series of expts. with stone from France, calcite, porphyry, basalt, quartzite and quartz treated similarly. pH of water in contact with calcite, porphyry, basalt and Orne quartzite increased, whereas pH of water in contact with Manche quartzite and Manche quartz decreased after period of 1 wk. Initial pH value of water 6.36. pH values of water in contact with Manche quartzite and Manche quartz 5.50 after 34 days; pH value of water in contact with this quartz 3.97 after 485 days. As there was little alteration in pH value of distd. water kept in closed pyrex vessels for 16 mo., decrease in pH value must have been due to dissolution of acid radicals from quartz. Observations continued for 16 mo. on water in contact with basalt; basalt then washed and placed in fresh distd. water. pH values of this water slightly less after 1, 4 and 6 days than pH values previously obtained. Different pH values obtained with different samples of gravel. When, however, samples of gravel sorted into 3 types according to their color, no differences in pH value observed. Therefore considered that size of stones had effect upon pH values obtained. Expt. therefore made with Derby limestone ground and sorted according to size. pH values of water in contact with stone after given period decreased with increasing size of particle and remained relatively const. for each size of stone. In similar expt. with Guernsey granite pH values of water in contact with stone also decreased with increasing size of particle, although after 22 days there was little difference between pH values reached with largest (¼″ to ½″) and smallest (dust to 70-mesh) stones. pH value of water in contact with smallest size, however, showed higher max. and reached max. earlier. In expt. with Orne quartzite, results obtained more erratic; pH values of water in contact with largest (¼″ to ½″) and smallest size (dust to 70-mesh) higher throughout expt. than those of water in contact with stones of intermediate sizes. After Orne quartzite had been soaking for 450 days, washed and soaked in fresh distd. water for 4 days. pH values obtained after soaking in second water lower after soaking for 3 hr. and longer than were those obtained after soaking in original water; these differences more pronounced with larger stones. Differences in pH values thought to be due to fact that more soluble salts removed during first soaking. Concluded that coatings on stones of ¼″ to ½″ size more soluble than coatings on smaller pieces of stone obtained by

crushing ¼″ to ½″ pieces. In order to det. whether washing of stones before soaking had any effect on pH values obtained, ¼″ to ½″ Orne quartzite divided into 2 portions. 1 portion washed in distd. water before soaking; other portion freed from dust by blowing before soaking. pH values observed during period of 3 days consistently higher with unwashed than with washed stone. In order to det. whether Derby limestone contained alkali, pH values obtained when limestone in contact with water compared with those obtained when pptd. calcium carbonate and "Analar" calcium carbonate, which contains not more than 0.1% alkali, in contact with water. pH value of water in contact with "Analar" calcium carbonate varied between 7.78 and 7.87. pH values of water in contact with pptd. calcium carbonate and with Derby limestone considerably greater; therefore concluded that these 2 substances contain more than 0.1% alkali. From nature of pH values obtained and conditions under which stones exist, considered that these alkalis probably sodium or potassium carbonate or calcium hydroxide. Anals. made of water which had been in contact with various stones; radicals detected in each case shown in table. Amts. of sodium, potassium and carbonate dissolved from granite, porphyry and gravel appear to be sufficient to acct. for pH values found. pH values of water in contact with limestones must be due largely to calcium hydroxide produced by hydrolysis of calcium carbonate or calcium silicate.—*W.P.R.*

Polarigraphic Determination of Chlorides, Bromides, Iodides, and Cyanides. S. G. Mikhlin. Trudy Vsesoyuz. Konf. anal. Khim. (U.S.S.R.) **2**: 507 ('43). Polarigraphic detn. of chloride, bromide, iodide and cyanide, using dropping-mercury electrode, discussed. Results showed that by adding small quant. of sulfur to electrolyte, which was 0.1*N* soln. of potassium nitrate, curve obtained on which waves corresponding to cyanide, bromide and iodide registered. Cyanide can be detd. by anodic polarization in soln. contg. 1000 times as much chloride, 50–60 times as much bromide, and 30–40 times as much iodide as cyanide. No interference caused by presence of sulfate, phosphate or carbonate ions in soln., which should be neutral or slightly acid. Iodide, bromide, and chloride can be detd. in same sample b[y] anodic polarization, provided that concn. [o]f each of these ions in soln. similar to that [o]f cyanide ion. If soln. contains considerabl[e] excess of chloride compared with bromide an[d] iodide, it must be dild. until concn. of chlorid[e] between 0.01 and 0.001*N;* chloride then detd by anodic polarization using galvanometer wit[h] low sensitivity. Such galvanometer will no[t] register iodide and bromide ions at concns. a[t] which they are present in dild. soln., but the[se] ions can be oxidized to iodate and bromate i[n] which form they may be detd. by cathodi[c] polarization. Detn. of iodate and bromate ca[n] be effected regardless of relative concns. of the[se] ions since there is considerable difference in the[ir] reduction potentials. Presence of chlorid[e], chlorate and perchlorate ions in soln., even i[n] relative concn. as great as 1,000,000:1, does no[t] affect detn. of iodate and bromate.—*W.P.R.*

Polarigraphic Determination of Small Quantities of Copper, Bismuth, Lead, Cadmium and Zinc in Natural Waters. A. A. Rezn[i]kov. Trudy. Vsesoyuz. Konf. anal. Khim. (U.S.S.R.) **2**: 573 ('43). Polarigraphic detn. of copper, bismuth, lead, cadmium and zin[c] in natural waters discussed. Conditions re[-]quired for satisfactory polarigraphic detn. of these elements are as follows: for copper soln. may be acid or alk. but no causti[c] alkalis must be present; for bismuth, eithe[r] acid soln. with pH value of less than 2 o[r] soln. contg. sodium potassium tartrate an[d] having pH value of 6 should be used; for lead any acid or alk. soln., except those contg. ammonium hydroxide, may be used; fo[r] cadmium, acid soln. with pH value of les[s] than 4, or soln. of ammonium hydroxid[e] should be used; for zinc, solns. with pH value greater than 1.5 may be used with exceptio[n] of alk. solns. of carbonates. Bismuth, lead cadmium and zinc can be detd. polarigraphi[-]cally in presence of each other. Copper and bismuth can be detd. in presence of each othe[r] in solns. of potassium sodium tartrate, citri[c] acid or tartaric acid. Min. concns. at which copper and zinc, and bismuth, lead and cad[-]mium can be directly detd. in presenc[e] of each other given. Solns. contg. smalle[r] concns. of these elements should be conc[d]. by extracting metals with dithizone dissolve[d] in carbon tetrachloride. Conditions require[d] for extraction of each element described. Mean error in detns. was ± 5%.—*W.P.R.*